STRAIN MEASUREMENTS IN THIN SHEAR WEB

AIRCRAFT TYPE BEAMS

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PREFACE

MEANING OF SYMBOLS USED

a	Long side of web panel, inches (distance between inner edges of chords in this report)
A_U	Actual cross-sectional area of upright, square inches
A_{U_e}	Effective cross-sectional area of upright, square inches
b	Short side of web panel, inches (distance between inner edges of uprights in this report)
B.S.F.	Bridge sensitivity factor of the SR-4 Wheatstone Bridge Control Box
C_R	Rivet correction factor, $\left(1 - \frac{\text{rivet diameter}}{\text{rivet pitch}}\right)$
C_1, C_2	Stress factors
E	Modulus of elasticity, psi (10.5×10^6 psi in this report)
G.S.F.	Gage sensitivity factor of the SR-4 strain gages
h_e	Effective depth of beam, centroid of top flange to centroid of bottom flange, inches
I_{NA}	Moment of inertia of area under consideration about its neutral axis, inches^4
k	Diagonal tension factor
S	Transverse shear force in web, inches
t	Thickness of web, inches (actual thickness in calculations)
t_U	Thickness of upright leg next to web, inches
σ_t	Primary diagonal tension stress in web on planes normal to buckles, ksi
σ_c	Primary compressive stress in web on planes parallel to line of buckles, ksi

σ_s' Secondary bending stress on planes normal to lines of buckles and at outer fibers of web, ksi

σ_s'' Secondary bending stress on planes parallel to lines of buckles and at outer fibers of web, ksi

σ_T Total outer fiber or maximum stress on planes normal to lines of buckles, ksi

σ_C Total outer fiber or maximum stress on planes parallel to lines of buckles, ksi

τ Nominal shear stress in web, ksi. In this report this stress is computed by the approximate formulae $\tau = S/1000h_e t$ where "t" is taken as the actual web thickness

τ_{cr} Critical shear stress, ksi

ϵ_t Elongation in direction parallel to buckles due to tensile stress σ_t acting alone, inches per inch

ϵ_c Contraction in direction perpendicular to buckles due to compressive stress σ_c acting alone, inches per inch

ϵ_s' Elongation or contraction in direction parallel to buckles due to secondary bending stress σ_s' acting alone, inches per inch

ϵ_s'' Elongation or contraction in direction perpendicular to buckles due to secondary bending stress σ_s'' acting alone, inches per inch

ϵ_T Total elongation in direction parallel to buckles due to σ_t and σ_c acting simultaneously, inches per inch

ϵ_C Total contraction in direction perpendicular to buckles due to σ_t and σ_c acting simultaneously, inches per inch

ϵ_{s_1} Total elongation or contraction in direction parallel to buckles due to σ_s' and σ_s'' acting simultaneously, inches per inch

ϵ_{s_2} Total elongation or contraction in direction perpendicular to buckles due to σ_s' and σ_s'' acting simultaneously, inches per inch

α Angle between axis of beam and direction of diagonal tension, degrees

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SUMMARY

Tests, both of pure shear and of cantilever beam shear web type panels of 24ST aluminum alloy, were made using electrical strain gages to determine the type and magnitude of stresses present. Both primary and secondary strains were measured at positions considered to be critical.

On pure shear panels of .025 and .040 nominal gage thickness, primary axial and secondary bending strains parallel and perpendicular to buckles were measured both at the center and at the diagonal tension corners. On two .064 nominal gage thickness pure shear panels, similar strains were measured, but only at the center of the panels.

In order to determine the effect of a/b ratio on total outer fiber stress, .040 nominal gage thickness panels of 1.0, 1.185, 1.455, and 1.882 a/b ratios were loaded in a centilever beam and primary and secondary strains were measured. Only for panels having an a/b = 1.0 were these strains measured both at the center and at the tension corner of the panel; all the rest had strains measured at the center only.

From the strains measured in these tests, the stresses present were calculated taking into account the effect of Poisson's ratio. The stresses thus obtained were: the primary tension and the secondary bending stresses on planes normal to the buckles, the primary compressive and the

secondary bending stresses on planes parallel to the buckles, and the total outer fiber or maximum stresses on these two planes which results from a superposition of the stresses acting on the plane under consideration.

Curves were plotted of the above stresses for each panel tested and two families of curves were drawn to show the variation of outer fiber stresses to a/b ratio.

Finally, the theoretical primary stresses were calculated by the latest theory on incompletely developed tension field beams and plotted along with experimental data as a family of curves for comparison.

INTRODUCTION

Although the last published theory[1] by the National Advisory Committee of Aeronautics for calculating primary stresses in buckled webs is supported by considerable amount of experimental data and gives excellent results in the elastic range, there is no evaluation given of the secondary stresses caused by the formation of buckles. And with the exception of the limited evaluation as given by Wagner[2] in an old report, there seems to be little other information regarding these stresses. Further related information was obtained by Kouns[3] who in making tests on panels with access holes

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[1]Kuhn, Paul and Chiartio, Patrick T., "The Strength of Plane Web Systems in Incomplete Diagonal Tension," Wartime Report L-367, N.A.C.A. ARR, August 1942.

[2]Wagner, Herbert and Lahde, R., "Tests for the Determination of the Stress Condition in Tension Fields," Technical Memorandum No. 809, N.A.C.A., 1936.

[3]Kouns, John H., "Electric Strain Gage Analysis of Stress Concentration in Shear Panel With An Access Hole," Thesis, Georgia School of Technology, Atlanta, Ga., 1947.

evaluated the secondary stresses; however, most authors indicate the difficulty of obtaining experimental values of these stresses and make no effort to evaluate them.

The results of tests with Stresscoat brittle lacquers done by McKee[4], and Mills[5] indicated a definite possibility of obtaining these secondary stresses since it was observed that, on buckling, the pattern of cracks formed in Stresscoat for a given load covered a reasonably large area and also gave some indication of the planes on which the secondary stresses combined with the primary stresses to produce maximum outer fiber stresses.

The purpose of this report is to determine more precisely the quantitative value of strains present in buckled webs of different a/b ratios and present the corresponding stresses.

Although the evaluation of strains beyond the yield point of the web may be made using strain gages provided the yield of the gage itself or the adhesive attaching the gage is not exceeded, the stresses themselves cannot be evaluated without a knowledge of the modulus of elasticity in the plastic range of the web material itself. Therefore, since the loading in the panel is so complex and since the evaluation of moduli in the plastic range for such a case would be a problem in itself, in this report, the strains in the plastic range were converted into stresses by the use of the elastic modulus of elasticity and are, therefore, nominal stresses.

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[4]McKee, William H., "An Investigation of Stresses in a Shear Panel with Access Hole by the Use of Stresscoat," Thesis, Georgia School of Technology, Atlanta, Ga., 1947.

[5]Mills, Frank C. Jr., "The Application of Stresscoat in the Study of Stresses in the Web of an Incompletely Developed Tension Field Beam," Thesis, Georgia School of Technology, Atlanta, Ga., 1947.

TEST EQUIPMENT

Jigs: The jig used for the pure shear panel tests, Figure 1, was the identical one used by Kouns[6] and similar to that used by Kuhn[7] at N.A.C.A. in tension field investigations. Four sets of 2 x $2\frac{1}{4}$ steel angles placed back to back and bolted to 12 inch square webs with 10-32 bolts at 1 inch on centers in staggered rows formed square panels having $8\frac{1}{4}$ inches clear width between flanges. Half inch bolts, 10 inches from center to center, formed the hinge points of the adjacent angles allowing the jig to distort into rhombic form under load. Pure shear loading was obtained by a link arrangement from two diagonally opposite corners.

For the tests on thin panels loaded in transverse shear, a cantilever beam shown in Figure 2 was used. Chord members were made up of two 3/16 x$1\frac{1}{4}$ x $1\frac{1}{4}$, 24ST extrusions placed back to back on each side of the web and capped over with 3/8 x $2\frac{1}{2}$ low carbon steel strips. One exception to this was for the beam test shown in Figure 5 where the steel cap strips were only 1/8 inch thick. Stiffeners made from 1/16 x 3/4 x 1, 24ST bulb extrusions and separated from the web by 3/16 x 3/4, 24ST filler strips were placed back to back on the web. Bolt attachments were as shown in Figure 2. All bolts were 1/4 inch aircraft bolts with the exception of those at the mounting point, where 3/16 inch bolts were used to attach the web. Five-sixteenth inch bolts were used to attach the chord members to the face plates. Provision was made for moving the two stiffeners next to

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[6]Kouns, op. cit.

[7]Kuhn, Paul, "Ultimate Stress Developed by 24ST Sheet In Incomplete Diagonal Tension," Technical No. 833, N.A.C.A., December 1942.

the loading plate inboard in $1\frac{1}{4}$ inch steps to obtain varying a/b ratios on the center panel and yet maintain an a/b = 1.0 on each side of this panel.

As the first run indicated the need, two parallel angle iron bars were used on each side of the beam on all subsequent runs to prevent torsional instability. These are shown in Figures 6 through 9.

Specimen: Pure shear panel specimens were made of 12 x 12 inch, 24ST sheet. Nominal gage thicknesses of .025, .040, and .064 were used and duplicate specimens were taken from the same sheet. Cantilever beam panel specimens were made of $10\frac{1}{2}$ x 42 inch, 24ST sheet. All specimens were of .040 nominal gage thickness and from the same stock with the grain structure parallel to the uprights.

Strain Gage System: All tests were made with standard SR-4 type electric strain gages manufactured by the Baldwin Locomotive Works, Baldwin Southwark Division.

A Baldwin Southwark SR-4 Wheatstone Bridge Control Box[8], schematically shown in Figure 3, was used to measure the change in resistance of the strain gages due to strain.

To measure primary stresses, two directly opposite gages on the web were connected in series and then into one arm of the bridge (terminals A and C, Figure 3) and two series connected temperature compensating gages, identical to the active gages, were connected into the opposite arm of the bridge (terminals C and B, Figure 3).

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[8]Anonymous, "SR-4 Bonded Metalectric Strain Gage," Bulletin 164-X, The Baldwin Locomotive Works, Baldwin Southwark Division, Philadelphia, Penn., 1941.

Secondary strain readings were obtained by connecting two gages, which were located directly opposite on the web, into adjacent arms of the control box (i.e., one gage in arm AC and the other in arm CB, Figure 3).

To facilitate changing circuits, two gang switches were used, Figure 3. A Mallory four gang switch, type 1335L, with heavy silver plated contacts was used for obtaining secondary strains, and a Kelvin switching box for obtaining primary strains. One box was always in the neutral position while the other was in operation. Current for the electrical system was supplied by two 6 volt dry cell batteries.

Loading Apparatus: A Riehle Universal Testing machine was used to apply load to the pure shear panels. To load the cantilever beams, a hydraulic jack was used on all tests except the first, where an ordinary screw jack was used. Accurate loading increments were obtained by using a calibrated Morehouse proving ring between the jack and the beam. Figure 4 shows the testing machine and Figures 5 through 9 show the jacks and the proving ring.

TEST PROCEDURE

For the pure shear tests, 12 x 12 inch panels of the desired gage thickness were cut, placed in the jig, and back drilled for the 3/16 bolts to provide alignment. Gages were mounted with Duco cement[9], one in line and the other normal to the expected buckle on both sides of the panel. The gages normal to the buckle were folded back on themselves giving them an effective length of $\frac{1}{2}$ inch. This shorter gage length tended

[9]Anonymous, "The SR-4 Bonded Resistance Wire Strain Gage," Bulletin No. 179, The Baldwin Locomotive Works, Baldwin Southwark Division, Philadelphia, Penn., 1945.

to give more nearly the maximum stress value. Gage terminals were attached in circuit as shown in Figure 3.

The jig was placed in the jaws of the testing machine and loads applied in equal increments through the link arrangement at diagonally opposite corners of the jig. Readings were taken at zero load and after each increment of load had been added and the loading system had stabilized. The upper limit of load applied was that considered to be sufficient to produce permanent set of the panel. This was taken to be anywhere from two to three times the critical buckling load and proved to be sufficient since permanent strain was read in every case when the zero reading was checked after releasing the load.

For the transverse shear beam panels, the preparation for runs and the actual test procedure were identical to that for the pure shear panels with two exceptions. First, the panels themselves in this case were all predrilled simultaneously. Secondly, the beam was mounted as a cantilever and the load was applied through the proving ring by means of jacks under the loading point. Loading increments for all the beams tested were 300 pounds and, in order to produce permanent set in the web, the maximum load was again two to three times the critical buckling load for the center panel of the beam tested.

Guide bars to prevent twisting of the beam, as shown in Figure 6 through 9, were used after the first run, since torsional instability occurred due to lack of lateral support for the flanges. Subsequent runs proved the modification entirely adequate.

DISCUSSION

The total strains set up in the outer fibers of a buckled web are compound in nature, being made up of superimposed strains caused by biaxial loading. The total primary strain in a given direction may be considered as made up of a strain due to a stress in the given direction and a strain due to a stress in the direction normal to the specified direction.[10] Similarly, the total secondary bending strain in a given direction due to the bending out of the web is made up of two parts: strain due to a bending stress in line with the specified direction and a strain due to a secondary stress normal to the specified direction. In each case, the strains due to stresses normal to the specified strain direction are the result of Poisson's ratio effect. Using the symbols of the preface, the above may be expressed algebraicly for total strains respectively parallel and normal to the buckle thus:

$$\epsilon_T = \epsilon_t - \mu\epsilon_c$$

$$\epsilon_C = \epsilon_c - \mu\epsilon_t$$

$$\epsilon_{s_1} = \epsilon_s' - \mu\epsilon_s''$$

$$\epsilon_{s_2} = \epsilon_s'' - \mu\epsilon_s'$$

Combining the equations, ϵ_t, ϵ_c, ϵ_s', ϵ_s'', and the corresponding stresses, σ_t, σ_c, σ_s', σ_s'', are obtained thus:

$$\epsilon_t = \frac{\epsilon_T + \mu\epsilon_C}{(1-\mu^2)}$$

and $$\sigma_t = \frac{E(\epsilon_T + \mu\epsilon_C)}{(1-\mu^2)}$$, etc.

Experimentally the total strains are obtainable from the change in readings of the SR-4 Wheatstone Bridge Control Box. These are

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[10] Timoshenko, S. and Mac Cullough, Gleason, H., Elements of Strength of Material, (D. Van Nostrand Co., Inc., New York, Second Edition, May 1940), p. 70.

$$\epsilon_T = \Delta M_1 \frac{B.S.F.}{G.S.F.} \quad \text{and} \quad \epsilon_C = \Delta M_2 \frac{B.S.F.}{G.S.F.}$$

$$\text{and} \quad \epsilon_{S_1} = \frac{\Delta M_3}{2} \frac{B.S.F.}{G.S.F.} \quad \text{and} \quad \epsilon_{S_2} = \frac{\Delta M_4}{2} \frac{B.S.F.}{G.S.F.},$$

where ΔM_1, ΔM_2, etc., are the changes in micrometer readings. The subscripts for the ΔM readings denote that they are different quantities. Dividing the ΔM readings by two is due to the doubling effect of having active gages in two arms of the bridge circuit instead of one. It is to be noted that the stresses as obtained by the above procedure will be merely nominal stresses in the plastic range of the web material since the value of E diminished at stresses above the proportional limit. Test values for beams tested, however, go only three to six hundred pounds above the value corresponding to the tensile or compressive yield of the outer fibers of the sheet, that is, yield values of 40000 psi. tension or compression, which corresponded here to a shear stress of approximately 15000 psi. Therefore, the values of the stresses shown, up to a shear stress of 15000 psi., may be considered as actual stresses.

Peterson[11] presented test data on a series of 25 inch depth beams which indicated that the primary stresses in the tension corners of buckled panels may be considerably greater than those at the center of the panel. Since it might also appear that at the corner the radius of curvature of the buckle, at least parallel with the buckle, might be greater than that at the center, the stresses might add together to give a greater outer fiber stress there than at the center. But such was not the case in any of

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[11]Peterson, James P., "Strain Measurements and Strength Tests of 25 inch Diagonal Tension Beams With Single Uprights," Wartime Report L-104, N.A.C.A., ARR No. L-5J02a, 1945.

the tests made for either pure shear or cantilever beam panels.

The principal information obtained from the light .025 pure shear panel tests, Figures 10 through 13, was that of a test procedure. It was concluded that thicker gage panels should be used so that the percent error in load readings would have less effect on corresponding strain readings. The chief cause of the load error was due to sluggish functioning of the hydraulic valves in the testing machine, making it difficult to hold the load steady long enough to get accurate strain readings. The guaranteed accuracy of the testing machine itself is low in this range. However, a worthwhile observation was made that reloading a prestressed panel may cause it to buckle in a different way. Such was the case for the panel of Figures 12 and 13, which buckled so that the crest of the buckle moved $\frac{1}{2}$ inch away from its original position under the gages. Even so, the stress values resulting were not far different from those of the previous test.

From the two tests on the .064 nominal gage panels, Figures 14 through 17, confirmation of the statement made by Kouns[12] regarding load reversals was observed. At $\tau = 15330$ psi., where the shear stress was suddenly increased to 18660 psi., dropped to 5328 psi. and then returned to 15330 psi., and at $\tau = 21300$ psi., where the load was held constant approximately 15 minutes, there is a noticable increase in strain and corresponding stress. These separate tests indicated that two identical stress panels would give practically identical stress results.

The pure shear test data obtained on the .040 nominal gage panel,

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[12]Kouns, op. cit., p. 12

Figures 18 and 19, furnish comparative figures for the subsequent beam panel tests. Also indication was given that the stresses in the corners, both primary, secondary, and totals, would be less than those in the center.

The results of the two tests made on beam panels having an $a/b = 1.0$, Figures 20 through 23, show a maximum difference of outer fiber stresses of $3\frac{1}{2}\%$ parallel to the buckle (curves A and B, Figure 32); and 23% perpendicular to the buckle (curves A and B, Figure 33), occurring at $\tau = 12000$ psi. Excellent agreement is also observed between these results and the results obtained from the pure shear panel plotted as curve G, Figures 32 and 33. The results shown by Figures 20, 21, and curve A of 32 and 33 were from the beam which had the 1/8 inch steel cap strips. Those of Figures 22, 23, and curve B of 32 and 33 were from the beam with 3/8 inch cap strips. The latter buckled so that the crest of the buckle fell $\frac{1}{4}$ inch off the center of the gages running parallel to the buckle.

The gages for the first beam tested having an $a/b = 1.185$ were located by using the Stresscoat analysis data from Mills'[13] work. However, the buckles as they first formed fell so that the gages were directly between the buckles, but after increasing the load still further the buckle straightened up and came practically under the gages. As a result the values of outer fiber stresses on planes parallel and perpendicular to the buckles at a shearing stress of 15000 psi. were within 7% and 0%, respectively, of the values obtained from the second test with $a/b = 1.185$. The gages of the latter were 1/8 inch off the crest of the buckle at all times.

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[13]Mills, <u>op. cit.</u>

The results of the first test are plotted in Figures 24 and 25, and those of the second test in Figures 26 and 27. Reasonably small sidewise displacement of the gages from the crest of the buckles as was present in the second test seemed to have practically negligible effect on all stresses read except the secondary bending stress on planes parallel to the buckle. Even here the difference is small if the gages perpendicular to the buckles are lapping over the crest as they were in this instance.

The results of tests having an $a/b = 1.455$ and an $a/b = 1.882$, plotted in Figures 28 through 31, show nothing new except that for a given transverse shear there is a dropping off of all stresses with increasing a/b ratios. The dropping off was even more pronounced for the test having an $a/b = 1.882$ and started approximately at $\tau = 8000$ psi.

Comparative results of all the a/b ratios tested were plotted in Figures 32 and 33. For increasing values of a/b ratio and a given shear stress, τ, there is observed a definite tendency for the maximum outer fiber stress on planes normal to the buckles to fall off. The same tendency is observed in stresses on planes parallel to the buckle although it is less pronounced. This dropping off of outer fiber stresses for a given shear stress is caused in this instance by holding "a" constant and decreasing "b". As τ/τ_{cr} is decreasing rapidly for such a condition, the dropping off is probably more a function of τ/τ_{cr} than of a/b ratio alone. Comparing the results of Figures 32 and 33 together shows that $\sigma_T \cong \sigma_C$ in absolute value for a given shear stress, τ. However, confirming Kouns'[14] statement, the critical web condition is probably still that due to σ_T.

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[14]Kouns, op. cit., p. 10

In order to show how the experimental strains were converted to stress, Tables I through V were prepared using the results of the test run on the beam having the 1/8 inch steel cap strips on the chords and an $a/b = 1.0$. Theoretical values of the primary stresses were calculated by Kuhn's latest method[15] and the results for an $a/b = 1.0$ ratio were tabulated in Table VI of Appendix I.

To give a comparative overall picture of the experimental stresses obtained and the theoretical primary stresses calculated by Kuhn's method, Figure 34 was drawn showing the experimental primary stress values, σ_t, σ_c, as test points; the theoretical primary stress values σ_t, σ_c, as solid lines; and the experimental outer fiber stress values as dotted lines. It is observed that within the limit of buckling stress set up by Kuhn[16] (i.e., taking τ=12000 psi. as upper limit, which is the approximate value where the outer fiber stresses pass from the elastic to the plastic range) the theoretical stresses are very close in every case, having a maximum deviation of 1%. The theoretical compressive stresses, however, are as much as 100% too high for the $a/b = 1.882$ results, and average approximately 50% high for the other a/b ratios. In calculating the theoretical shear stress, since the ratio of upright thickness to web thickness, t_U/t, was greater than 3, all edges were considered clamped and the points for measuring panel widths and depths for a/b ratios were taken as the points where the sheet emerged from the stiffeners and chords as suggested by Kuhn[17].

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[15] Kuhn and Chiartio, <u>op. cit.</u>

[16] <u>Ibid.</u> pp. 18-19 and 28

[17] <u>Ibid.</u>, p. 13

Although Wagner's[18] predictions based on tests on brass sheets in 1936 have been questioned as being entirely too optimistic, his predicted percentage overstress of the outer fiber stresses over the stress midway between the two surfaces of the sheet for the center of the panel agree almost exactly with the results obtained in this report. However, the value of stresses in the median area of the sheet, that is, the primary stresses, that his theory predicts do prove to be slightly unconservative for design.

In view of the fact that the total outer fiber stresses in the panels tested were sufficiently high to be past the yield of the material and produce permanent set at limit loading conditions of $\tau/\tau_{cr} \cong 2$, it seems advisable to suggest that, if a panel is to be subjected to load reversals, caution be used in applying the results of this report to it, especially when the loading condition is such that design $\tau/\tau_{cr} > 3$. Though most of the present theories on incompletely developed tension field beams have little or no such limitation placed upon them by statements of their originators, it is believed that such limitations exist and should be specified.

CONCLUSIONS

The tests made indicated that strain measurements of secondary stresses as well as primary stresses can be made, and results with only slight scatter of points obtained provided care is used in locating strain gages. The effect of slight mislocation of gages with respect to the buckle crest produced practically unnoticeable effect on secondary stresses on planes

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[18] Wagner, op. cit.

parallel to the buckles and also reasonably small scatter of secondary stresses on planes perpendicular to the buckles.

Results also indicated that in the region of τ/τ_{cr} tested, the outer fiber stresses at the center of the panel would always be larger than at the corners and in most cases the primary stresses also would be larger. Furthermore, the results indicated that the maximum excess of the outer fiber stresses over the stresses on the median area of the web would be approximately 25 to 30% for the beam dimensions used; which agreed closely with values predicted by Wagner[19]. Such a relationship, if further supported by a larger number of tests, could be used for setting up an imperical relationship for obtaining total outer fiber stresses for design. Another consideration for setting up design expressions is that of load reversals and fatigue. Although this will also require considerable test data, the results obtained in this report indicated that serious web damage may be produced by load reversals if the design condition is such that $\tau/\tau_{cr} \geq 3$ (i.e., a limit loading condition of $\tau/\tau_{cr} \geq 2$). Thus, some such limiting criterion should be specified if the panel to be designed is subjected to load reversals.

The magnitude of primary, secondary, and outer fiber stresses at the center of the panel obtained also shows:

(1) a definite trend for the primary compressive stress to increase, the primary tension stress and the secondary stresses to decrease with increasing web thickness for pure shear panels with $a/b = 1.0$, due to higher critical buckling stresses;

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[19] Ibid.

(2) for a given thickness and a given shear stress, a definite tendency for all stresses, primary, secondary, and totals, to fall off with decrease in width, b;

(3) practically no difference in outer fiber stresses on planes parallel to the buckle for an $a/b = 1.0$ between pure shear panels and panels of beams with transverse shear load;

(4) a more pronounced difference in outer fiber stresses on planes perpendicular to the buckle for an $a/b = 1.0$ between pure shear panels and panels of beams with transverse shear load, the maximum difference being within 16%, which could probably be reduced with more test data.

The agreement of test results with theory for primary stresses agree closely with the comparisons made to test data on 40 and 25 inch depth beams made by N.A.C.A.,[20] thus indicating the reliability of the data herein presented on panels having larger thickness to beam depth ratios than N.A.C.A. tests.

Further tests should be made by using beams with greater depths and different gage thicknesses to confirm the conclusions drawn here on secondary stresses.

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[20] Kuhn and Chiartio, and Peterson, op. cit.

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APPENDIX I, Tables

TABLE I

PRIMARY DIAGONAL TENSION STRESS, σ_t , CENTER GAGES,

.039 24ST WEB, a/b = 1.0 Ref. Figs. 20 & 34

$E = 10.5 \times 10^6$ $\mu = .33$ G.S.F. = 2.04 $\tau = S/1000h_e t$ $\sigma_t = \dfrac{E(\epsilon_t + \mu\epsilon_c)}{1000(1-\mu^2)}$

LOAD	SHEAR STRESS 1000τ	DECADE	MICRO. READ'G.	ΔM	B.S.F. $\times 10^{-4}$	$\Delta\epsilon_t$ $\times 10^{-4}$	ϵ_t $\times 10^{-4}$	$\mu\epsilon_t$ $\times 10^{-4}$	$(\epsilon_t + \mu\epsilon_c)$ $\times 10^{-4}$	STRESS $1000\sigma_t$
0	0	5030	.535	0	2.395	0	0	0	0	0
300	756		1.336	.801		.943	.943	.311	.656	770
600	1512		2.140	1.605		1.890	1.890	.623	1.310	1540
900	2268		2.912	2.377		2.800	2.800	.923	1.945	2290
1200	3020		3.757	3.222		3.790	3.790	1.250	2.638	3100
1500	3775		4.580	4.045		4.750	4.750	1.567	3.309	3890
1800	4530		5.450	4.915		5.780	5.780	1.905	4.146	4870
2100	5290		6.510	5.975		7.030	7.030	2.310	5.210	6130
2400	6040		7.635	7.100		8.350	8.350	2.750	6.378	7500
2700	6800		8.960	8.425		9.900	9.900	3.265	7.725	9100
3000	7560		10.190	9.655		11.310	11.350	3.740	9.020	10600
3300	8320		11.480	10.945		12.900	12.900	4.250	10.380	12200
3600	9020		12.775	12.240		14.400	14.400	4.740	11.690	13760
3600	9020	4431	0	0	2.391	0				
3900	9830		1.340	1.342		1.580	15.980	5.270	13.060	15380
4200	10580		2.537	2.537		2.980	17.380	5.730	14.280	16800
4500	11330		3.740	3.740		4.400	18.800	6.200	15.480	18200
4800	12080		5.075	5.075		5.970	20.370	6.710	16.800	19800
5100	12830		6.375	6.375		7.500	21.900	7.230	18.080	21300
5400	13600		7.757	7.757		9.130	23.530	7.750	19.400	22800
5700	14350		9.072	9.072		10.670	25.070	8.270	20.630	24300

TABLE II

PRIMARY DIAGONAL COMPRESSIVE STRESS, σ_c , CENTER GAGES,

.039 24ST WEB, a/b = 1.0 Ref. Figs. 21 & 34

$E = 10.5 \times 10^6$ $\mu = .33$ G.S.F. = 2.04 τ = S/1000h_et $\sigma_c = \dfrac{E(\epsilon_c + \mu\epsilon_t)}{1000(1-\mu^2)}$

LOAD	SHEAR STRESS 1000τ	DECADE	MICRO. READ'G.	$-\Delta M$	B.S.F. $\times 10^{-4}$	$-\Delta\epsilon_c$ $\times 10^{-4}$	$-\epsilon_c$ $\times 10^{-4}$	$-\mu\epsilon_c$ $\times 10^{-4}$	$(-\epsilon_c - \mu\epsilon_t)$ $\times 10^{-4}$	STRESS 1000σ_c
0	0	6640	13.610	0	2.402	0	0	0	0	0
300	756		12.865	.745		.836	.876	.287	.565	-665
600	1512		12.113	1.497		1.760	1.760	.580	1.137	-1340
900	2268		11.405	2.205		2.595	2.595	.855	1.672	-1970
1200	3020		10.640	2.970		3.495	3.495	1.152	2.245	-2640
1500	3775		9.980	3.630		4.270	4.270	1.441	2.703	-3180
1800	4530		9.309	4.201		4.950	4.950	1.634	2.045	-3580
2100	5290		8.923	4.637		5.520	5.520	1.820	3.210	-3780
2400	6040		8.520	5.090		5.980	5.980	1.972	3.230	-3800
2700	6800		8.008	5.602		6.600	6.600	2.175	3.335	-3920
3000	7560		7.600	6.010		7.070	7.070	2.330	3.330	-3920
3300	8320		7.111	6.499		7.640	7.640	2.520	3.390	-3990
3600	9070		6.615	6.995		8.230	8.230	2.710	3.490	-4110
3900	9830		6.086	7.524		8.860	8.860	2.920	3.590	-4220
4200	10580		5.610	8.000		9.410	9.410	3.100	3.680	-4330
4500	11330		5.048	8.562		10.080	10.080	3.320	3.880	-4560
4800	12080		4.395	9.215		10.820	10.820	3.570	4.110	-4830
5100	12830		3.758	9.852		11.580	11.580	3.820	4.350	-5120
5400	13600		2.988	10.622		12.520	12.520	4.130	4.770	-5610
5700	14350		2.170	11.440		13.470	13.470	4.440	5.200	-6120

TABLE III

SECONDARY BENDING STRESS, σ_s', PARALLEL TO BUCKLE,

CENTER GAGES, .039 24ST WEB, a/b = 1.0 Ref. Fig. 20

$E = 10.5 \times 10^6$ $\mu = .33$ G.S.F. = 2.04 $\tau = S/1000h_et$ $\sigma_s' = \frac{E(\epsilon_s' + \mu\epsilon_s')}{1000(1-\mu^2)}$

LOAD	SHEAR STRESS 1000τ	DECADE	MICRO. READ'G.	$\frac{\Delta L}{2}$	B.S.F. $\times 10^{-4}$	$\Delta\epsilon_s'$ $\times 10^{-4}$	ϵ_s' $\times 10^{-4}$	$\mu\epsilon_s'$ $\times 10^{-4}$	$(-\epsilon_s' - \mu\epsilon_s')$ $\times 10^{-4}$	STRESS $1000\sigma_s'$
0	0	4569	1.440	0	2.495	0	0	0	0	0
300	756		1.511	.036		.044	.044	.015	-.026	31
600	1512		1.575	.078		.096	.096	.032	-.053	62
900	2268		1.645	.103		.126	.126	.042	-.021	25
1200	3020		1.753	.157		.192	.196	.063	-.051	-55
1500	3775		1.919	.240		.292	.292	.096	.020	-24
1800	4530		2.202	.391		.478	.478	.158	.178	-209
2100	5290		2.661	.612		.750	.750	.248	.650	-765
2400	6040		3.015	.788		.965	.965	.319	1.420	-1670
2700	6800		3.310	.935		1.144	1.144	.378	2.456	-2890
3000	7560		3.440	1.000		1.223	1.223	.404	3.187	-3750
3300	8320		3.551	1.056		1.291	1.291	.426	4.089	-4810
3600	9070		3.630	1.095		1.341	1.341	.443	4.684	-5500
3900	9830		3.700	1.130		1.384	1.384	.457	5.516	-6490
4200	10580		3.762	1.161		1.422	1.422	.470	6.248	-7330
4500	11330		3.810	1.185		1.452	1.452	.479	7.028	-8270
4800	12080		3.916	1.238		1.515	1.515	.500	8.025	-9430
5100	12830		3.990	1.275		1.561	1.561	.515	8.559	-10080
5400	13600		4.102	1.331		1.631	1.631	.538	9.289	-10920
5700	14350		4.200	1.380		1.688	1.688	.557	9.962	-11720

TABLE IV

SECONDARY BENDING STRESS, σ_s'', PERPENDICULAR TO BUCKLE, CENTER GAGES, .039 24ST WEB, a/b = 1.0 Ref. Fig. 21

$E = 10.5\times10^6$ $\mu = .33$ G.S.F. = 2.04 $\tau = S/1000h_e t$ $\sigma_s'' = \dfrac{E(\epsilon_s'' + \mu\epsilon_s')}{1000\,(1-\mu^2)}$

LOAD	SHEAR STRESS 1000τ	DECADE	MICRO. READ'G.	$-\frac{\Delta M}{2}$	R.S.F. $\times10^{-4}$	$-\Delta\epsilon_s''$ $\times10^{-4}$	$-\epsilon_s''$ $\times10^{-4}$	$-\mu\epsilon_s''$ $\times10^{-4}$	$(-\epsilon_s''-\mu\epsilon_s')$ $\times10^{-4}$	STRESS $1000\sigma_s''$
0	0	5130	11.650	0	2.50	0	0	0	0	0
300	756		11.561	.045		.055	.055	.018	.040	-47
600	1512		11.440	.105		.129	.129	.043	.097	-114
900	2268		11.131	.260		.318	.318	.105	.276	-325
1200	3020		10.935	.358		.438	.438	.145	.375	-441
1500	3775		10.105	.773		.945	.945	.312	.849	-1000
1800	4530		8.399	1.626		1.990	1.990	.656	1.832	-2155
2100	5290		4.715	3.468		4.250	4.250	1.400	4.002	-4710
2400	6040		-.165	5.908		7.230	7.230	2.385	6.911	-8130
2400	6040	6050	13.790	0	2.505	0				
2700	6600		7.810	2.990		3.670	10.900	3.600	10.522	-12400
3000	7560		3.767	5.012		6.140	13.370	4.410	12.966	-15220
3000	7560	6946	13.840	0	2.509	0				
3300	8320		9.019	2.411		2.955	16.325	5.380	15.899	-18800
3600	9070		4.810	4.015		4.910	18.280	6.025	17.837	-21000
3600	9070	8006	13.832	0	2.512	0				
3900	9830		9.530	2.151		2.650	20.930	6.900	20.473	-24100
4200	10580		5.790	4.021		4.960	23.240	7.670	22.770	-26750
4500	11330		1.805	6.014		7.420	25.700	8.480	25.221	-29700
4500	11330	9989	13.697	0	2.515	0				
4800	12080		9.515	2.591		3.195	28.895	9.540	28.395	-33400
5100	12830		5.610	4.044		4.980	30.680	10.120	30.165	-35400
5400	13600		1.420	6.139		7.430	33.130	10.920	32.592	-38300
5400	13600	L 7901	13.845	0	2.511	0				
5700	14350		10.341	1.752		2.160	35.290	11.650	34.733	-40800

TABLE V

TOTAL STRESSES PARALLEL AND PERPENDICULAR TO BUCKLE,

CENTER GAGES, .039 24ST WEB, a/b = 1.0 Ref. Figs. 32-34

SHEAR STRESS 1000τ	STRESS $1000\sigma_t$	STRESS $1000\sigma_s'$	TOTAL STRESS $1000\sigma_T$	STRESS $1000\sigma_c$	STRESS $1000\sigma_s''$	TOTAL STRESS $1000\sigma_C$
0	0	0	0	0	0	0
756	770	-31	739	-665	-47	-712
1512	1540	-62	1470	-1340	-114	-1454
2268	2290	-25	2265	-1970	-325	-2295
3020	3100	55	3155	-2640	-441	-3081
3775	3890	24	3914	-3180	-1000	-4180
4350	4870	209	5079	-3580	-2155	-5735
5290	6130	765	6895	-3780	-4710	-8490
6040	7500	1670	9170	-3800	-8130	-11930
6800	9100	2890	11990	-3920	-12400	-16320
7560	10600	3750	14350	-3920	-15220	-19140
8320	12200	4810	17010	-3990	-18800	-22790
9020	13760	5500	19260	-4110	-21000	-25110
9830	15380	6490	21870	-4220	-24100	-28320
10580	16800	7330	24130	-4330	-26750	-31080
11330	18200	8270	26470	-4560	-29700	-34260
12080	19800	9430	29230	-4830	-33400	-38230
12830	21300	10800	31380	-5120	-35400	-40520
13600	22800	10920	33720	-5610	-38300	-43910
14750	24300	11720	36020	-6120	-40800	-46920

TABLE VI

THEORETICAL STRESSES PARALLEL AND PERPENDICULAR TO BUCKLE FOR a/b = 1.0 CALCULATED BY KUHN'S METHOD, WR-L-367

Ref. Fig. 34

SECTION PROPERTIES:

$a/b = 1.0$, $I_{N_{A_{chord}}} = .201$ IN^4

$h_e = 10.15$, $I_{N_{A_{beam}}} = 84.6$ IN^4

$t = .039$, $A_{U_e} = A_U = .506$ IN^2

CALCULATED DATA: (By WR-L-367)

$\tau_{cr} = 3.650$ ksi, $C_2 = .22$

$\tau = .002525$, $C_R = 0$ at center

$C_1 = 0$, $\alpha = 45^\circ$, $A_{U_e}/dt = 1.62$

$$\sigma_t = \left[2\tau k + \tau(1-k)\right](1+kC_1)(1+kC_2)/C_R \qquad \text{--(1)}$$

$$\sigma_c = \left[\tau(1-k)\right](1+kC_1)(1+kC_2)/C_R \qquad \text{--(2)}$$

LOAD S	τ/τ_{cr}	k	1000 × τ	1+k	1+kC$_2$	1000 × σ_t	1-k	1000 × σ_c
0	0							
600	.414							
1200	.828							
1800	1.242	.11	4530	1.11	1.024	5150	.89	-4140
2400	1.656	.20	6040	1.20	1.044	7580	.80	-5060
3000	2.070	.34	7560	1.34	1.075	10900	.66	-5370
3600	2.485	.40	9070	1.40	1.088	13820	.60	-5930
4200	2.900	.44	10580	1.44	1.097	16700	.56	-6500
4800	3.310	.47	12080	1.47	1.104	19600	.53	-7080
5400	3.730	.50	13600	1.50	1.110	22700	.50	-7550
6000	4.140	.53	15140	1.53	1.117	25900	.47	-7970
6600	4.560	.56	16660	1.56	1.124	29300	.44	-8280
7200	4.960	.58	18150	1.58	1.127	32200	.42	-8690
7800	5.380	.60	19680	1.60	1.132	35700	.40	-8970

APPENDIX II, Figures

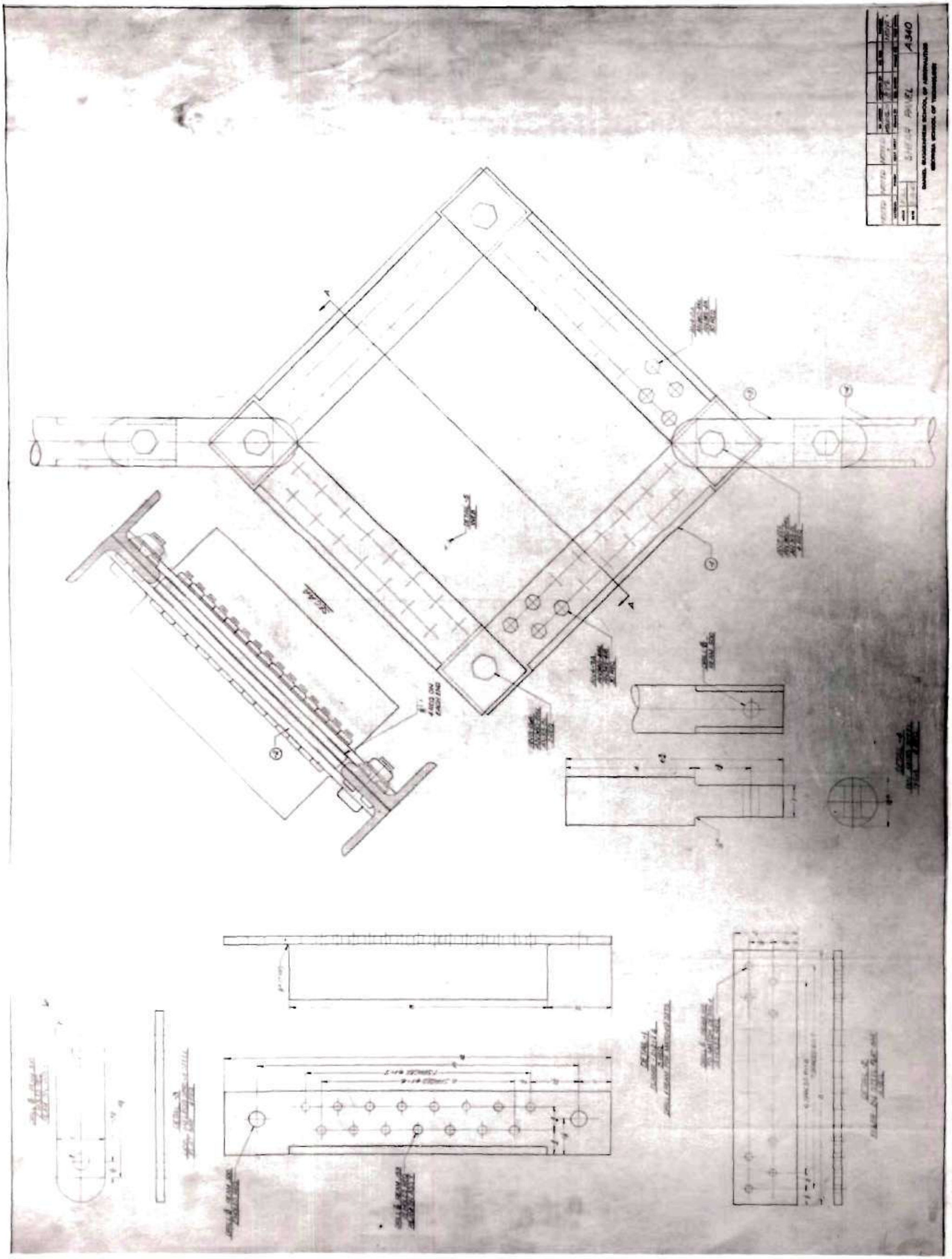


Figure 1.

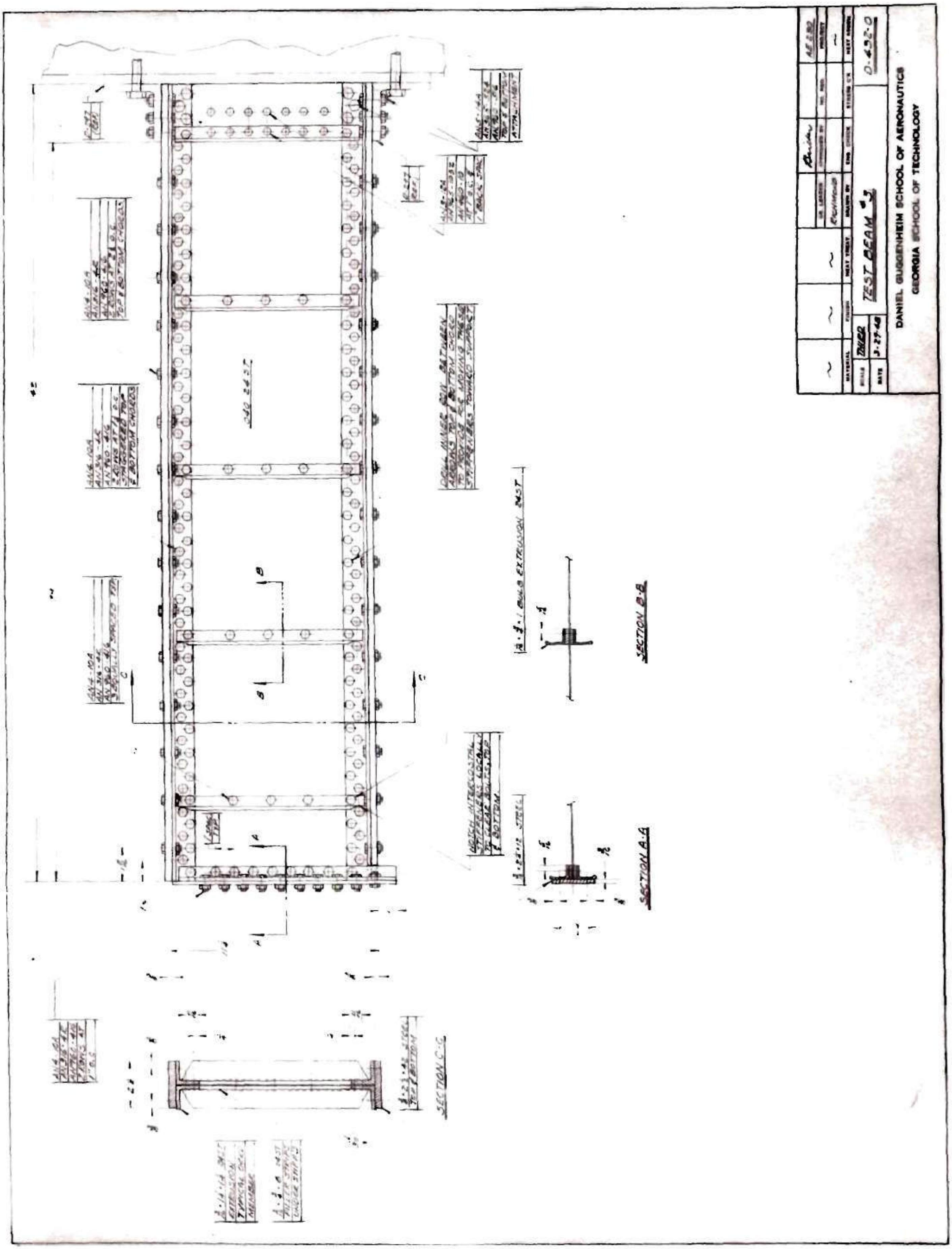
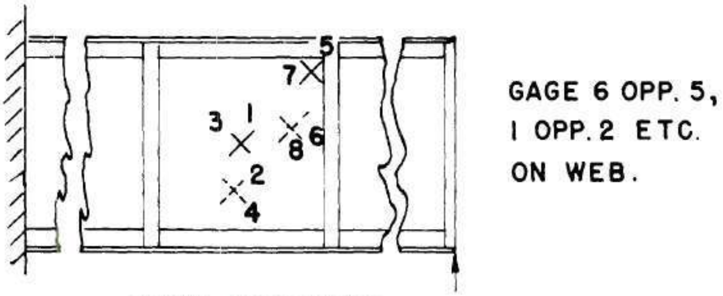
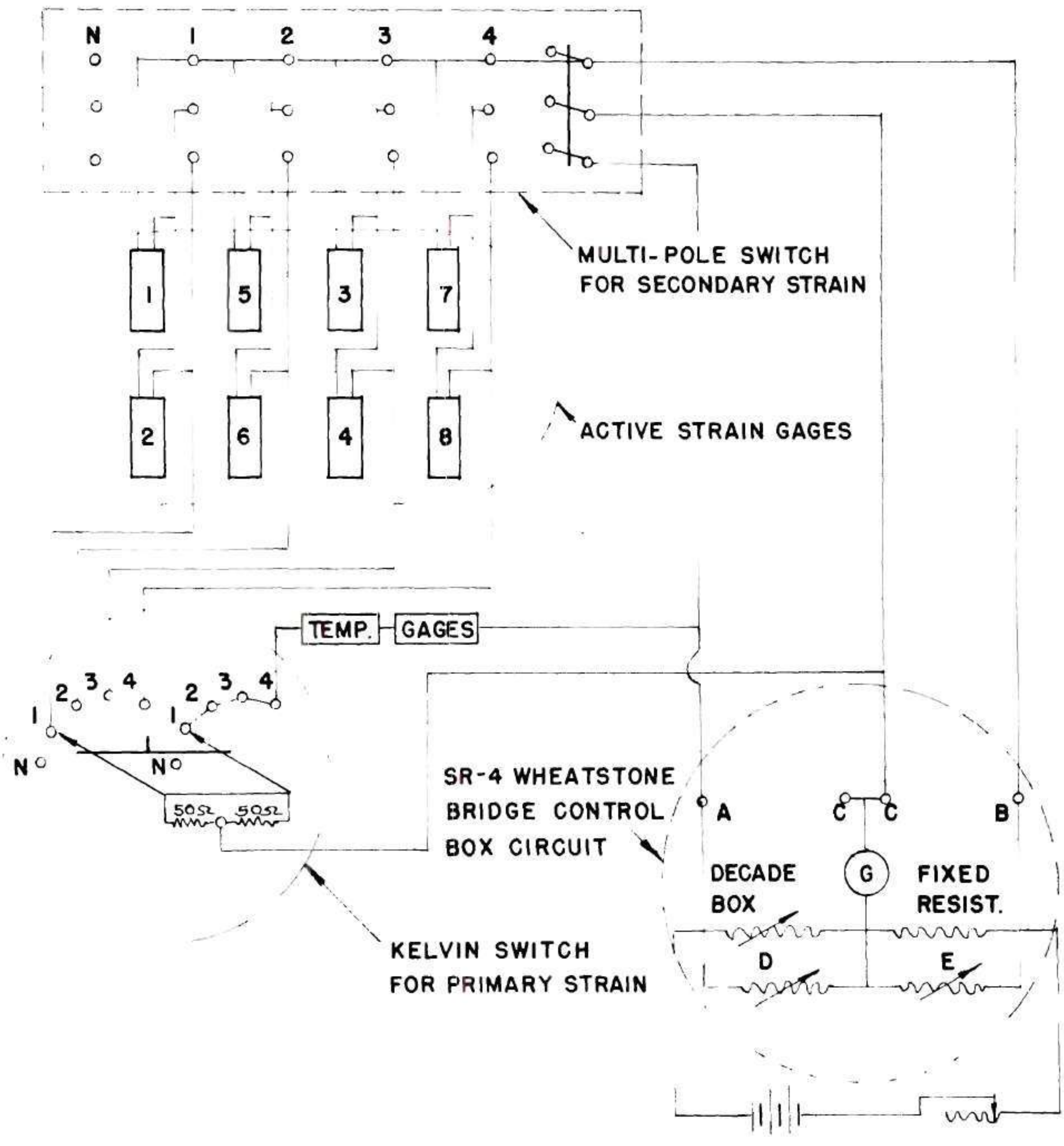

DANIEL GUGGENHEIM SCHOOL OF AERONAUTICS
GEORGIA SCHOOL OF TECHNOLOGY
TEST BEAM #3
D-432-0
SECTION A-A
SECTION B-B
SECTION C-C

Figure 2.



GAGE LOCATIONS



ELECTRICAL CIRCUIT USED
FOR MEASURING STRAINS

Figure 3.

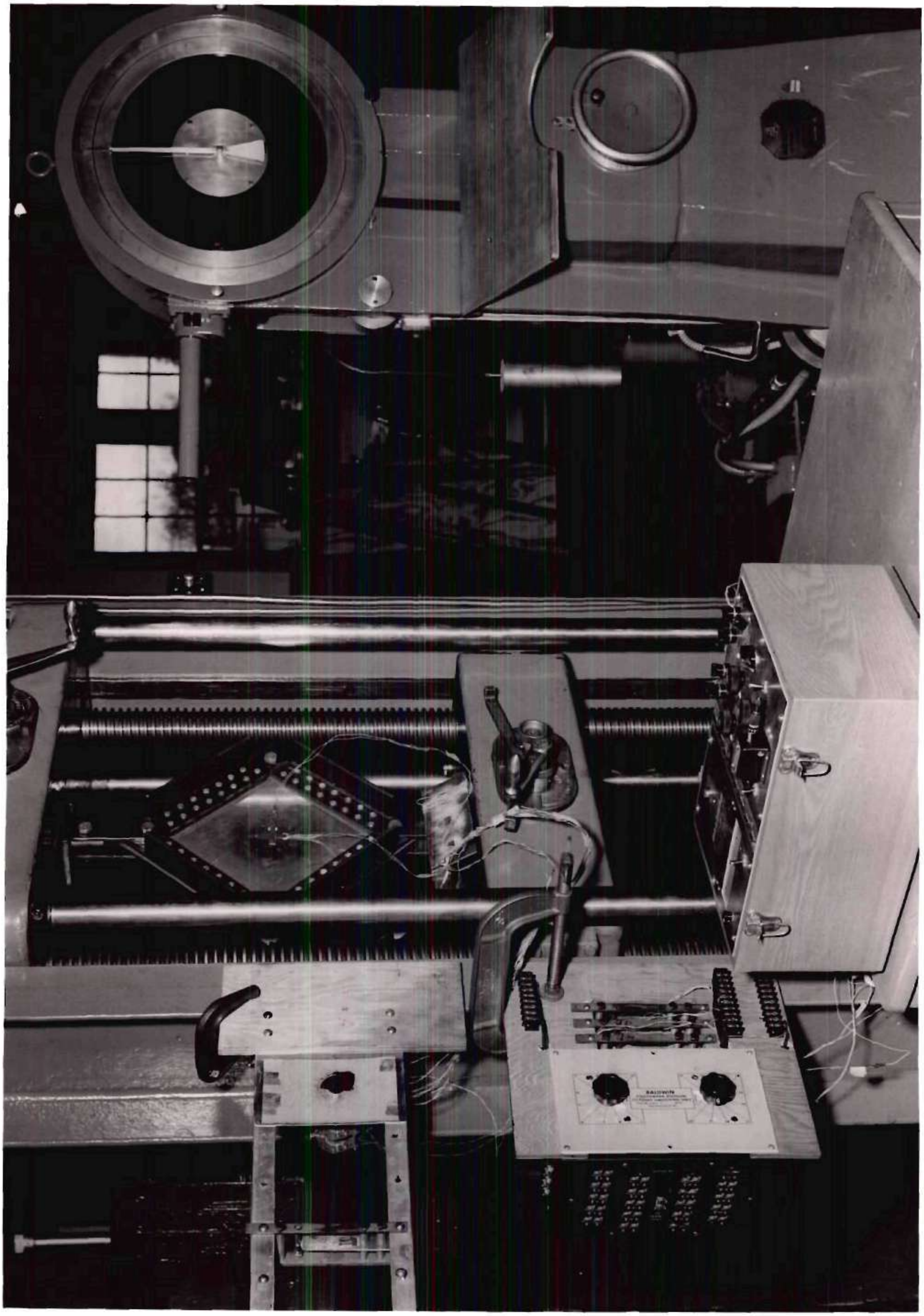


Figure 4. Pure Shear Panel In Testing Machine

Figure 5. Test Beam, a/b = 1.0, 1/8 Steel Cap Strips

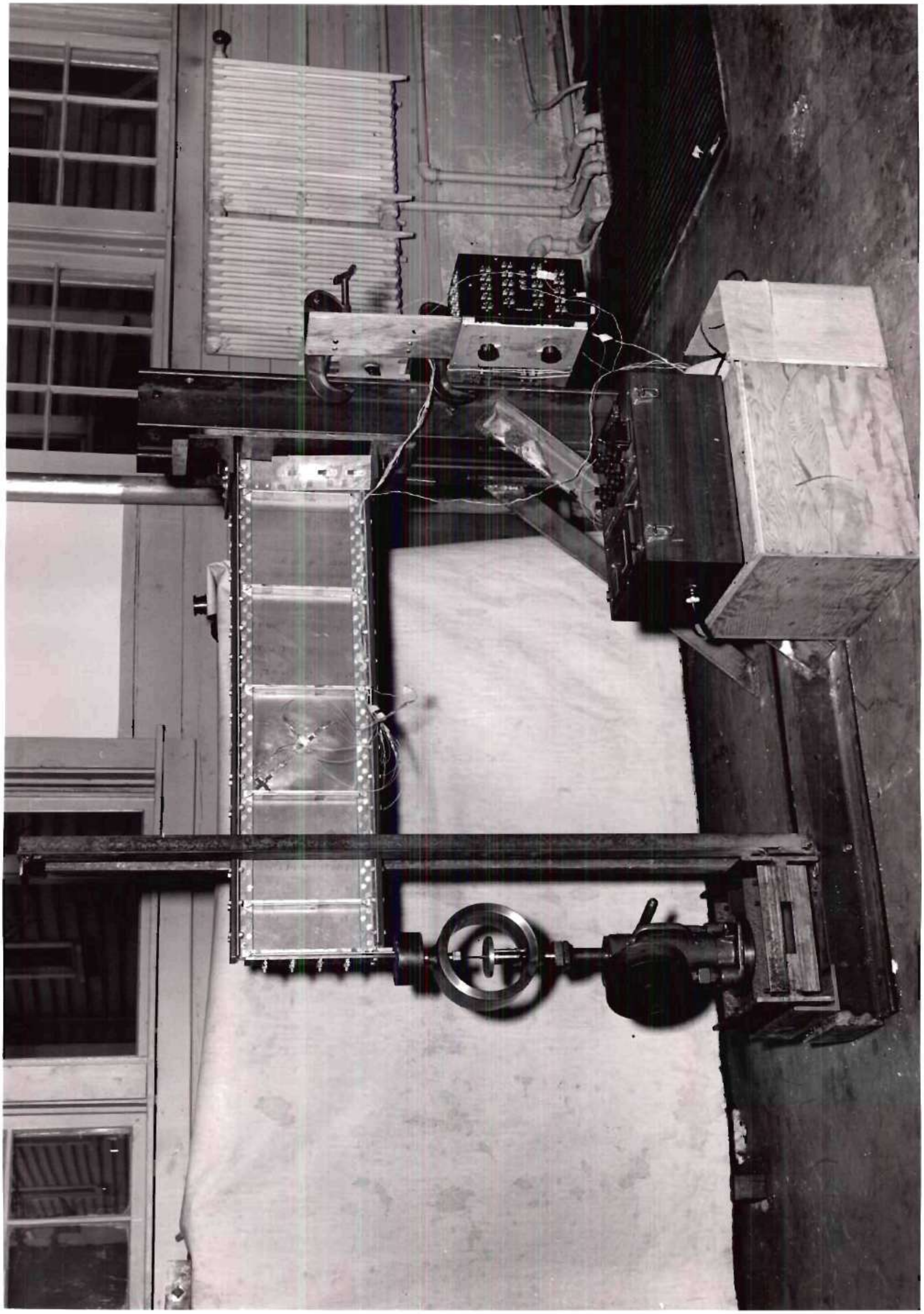


Figure 6. Test Beam, a/b = 1.0, 3/8 Steel Cap Strips

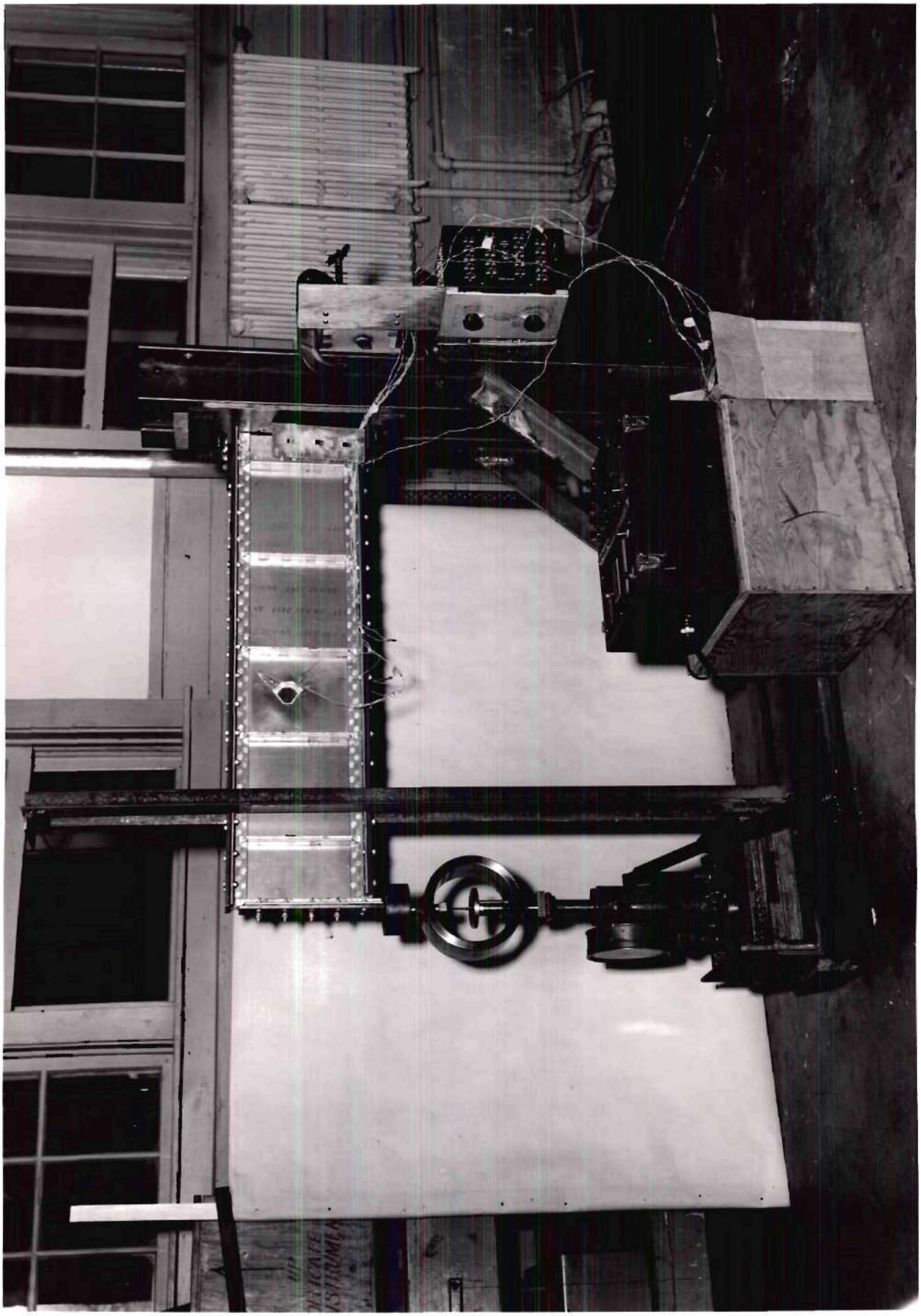


Figure 7. Test Beam, a/b = 1.185

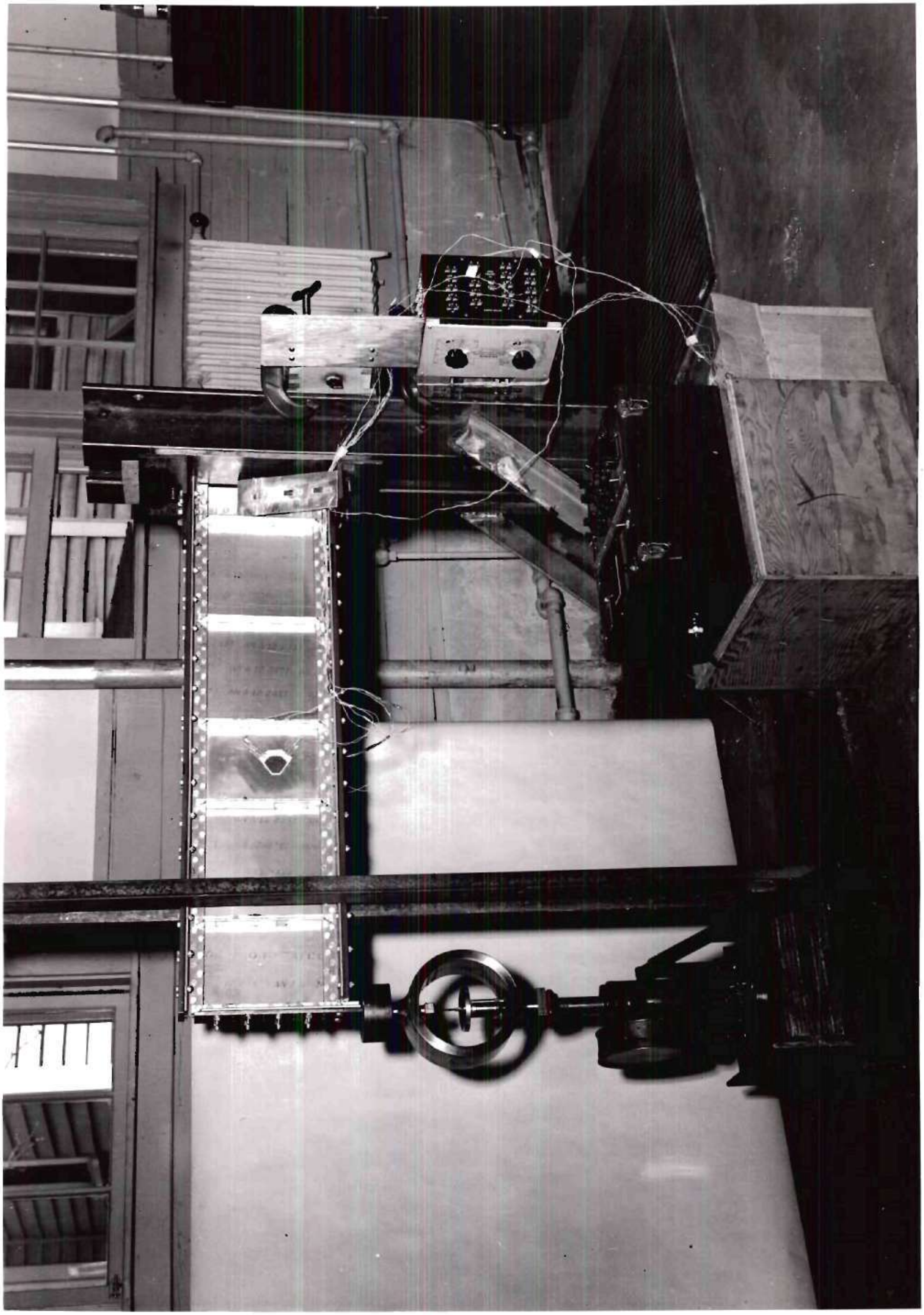


Figure 8. Test Beam, $a/b = 1.455$

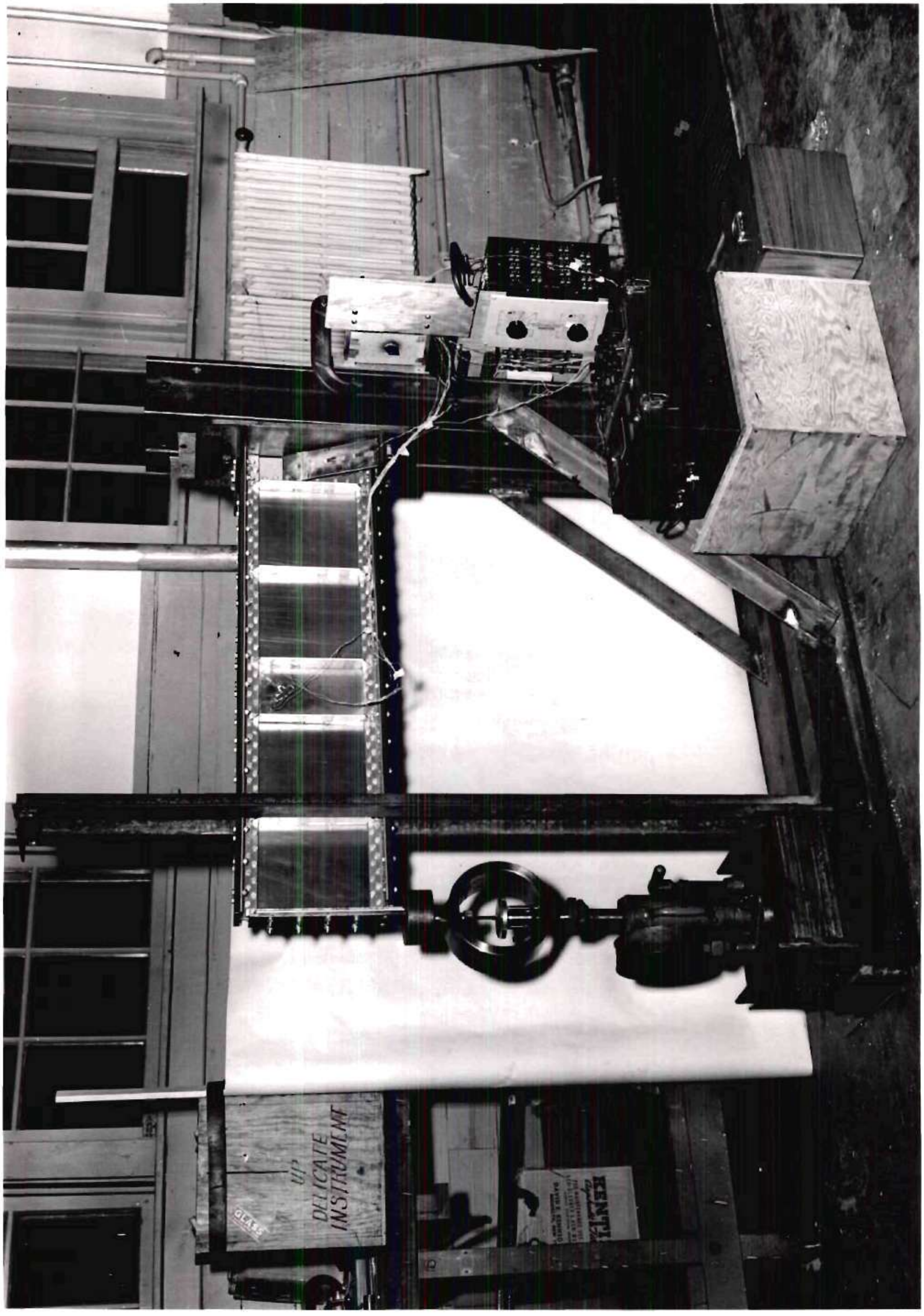
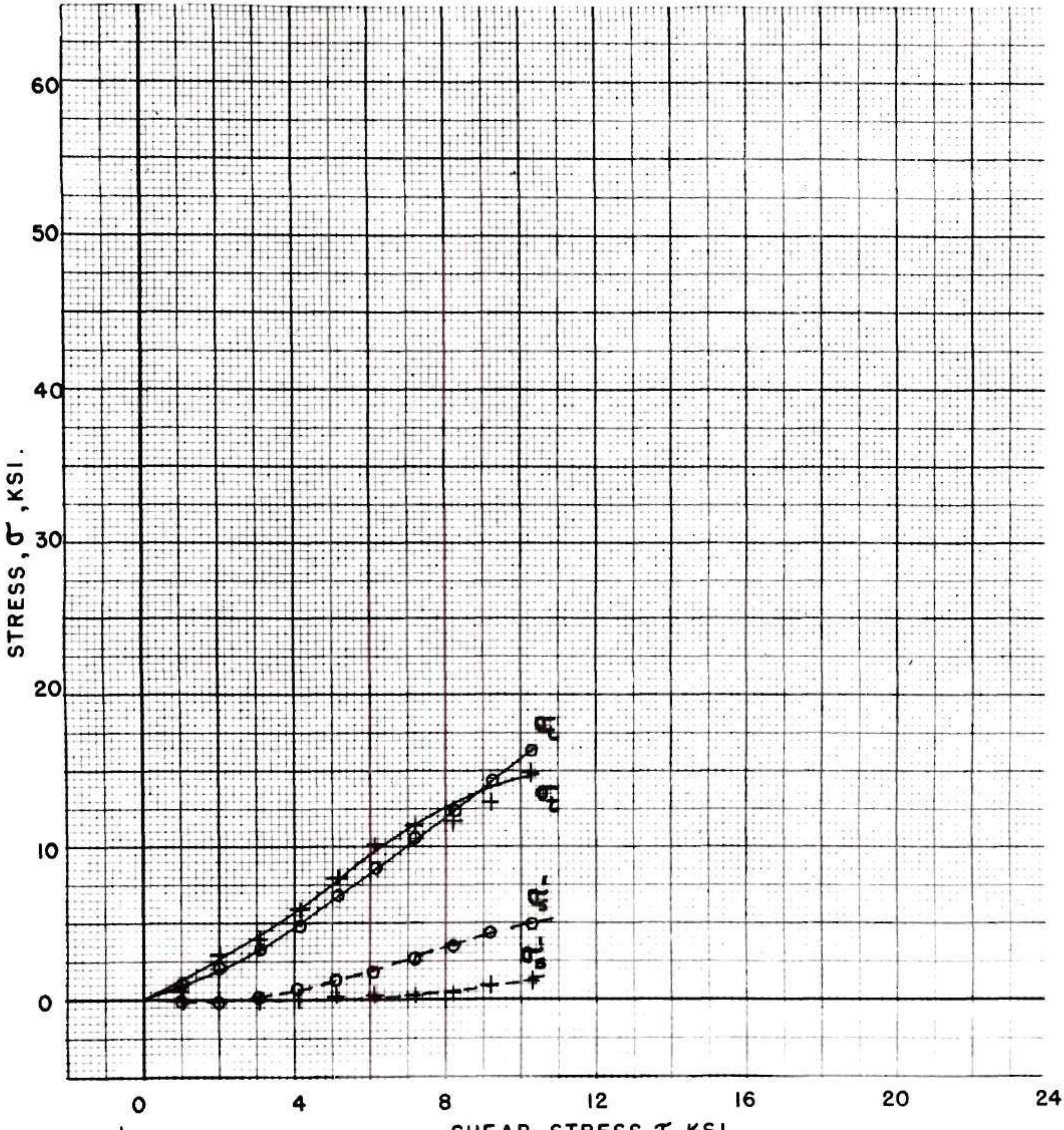
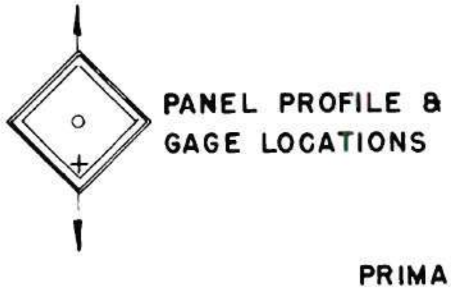


Figure 9. Test Beam, $a/b = 1.882$

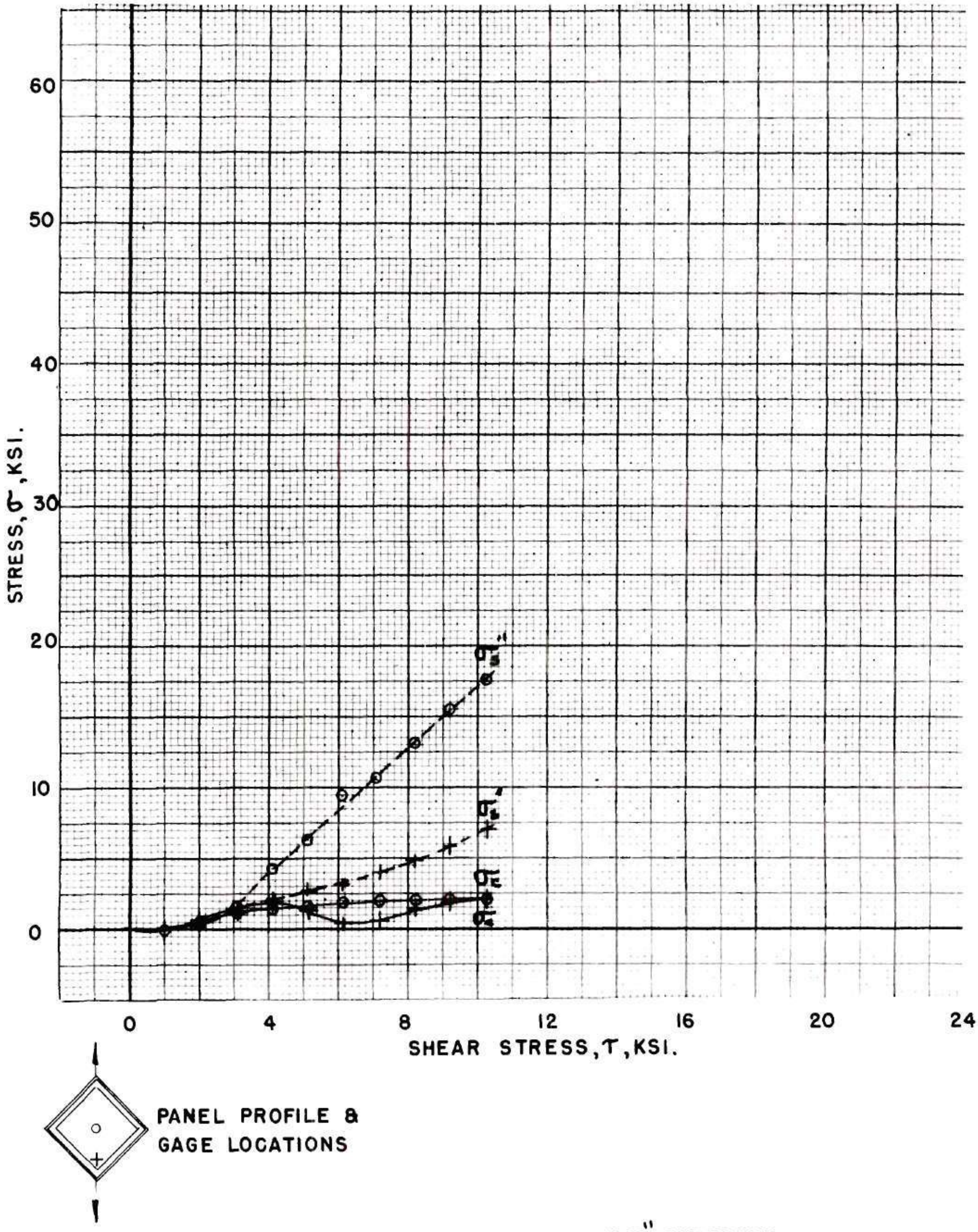




PRIMARY, σ_t, & SECONDARY, σ_s', STRESSES PARALLEL TO BUCKLE VS. SHEAR STRESS, τ.

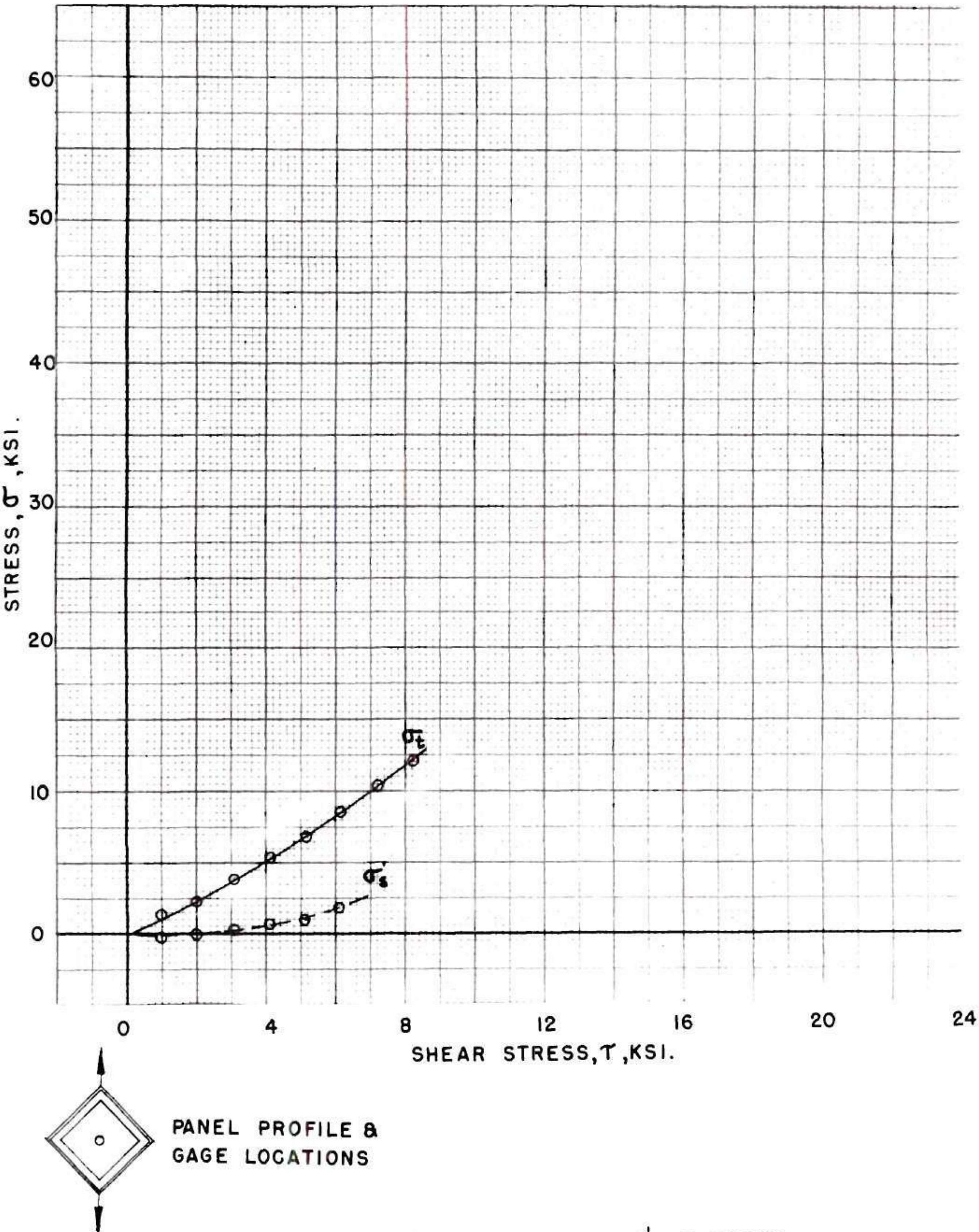
.026 24ST

Figure 10.



PRIMARY, σ_c, & SECONDARY, σ_s'', STRESSES
PERPENDICULAR TO BUCKLE VS. SHEAR STRESS, τ.
.026 24ST

Figure 11.

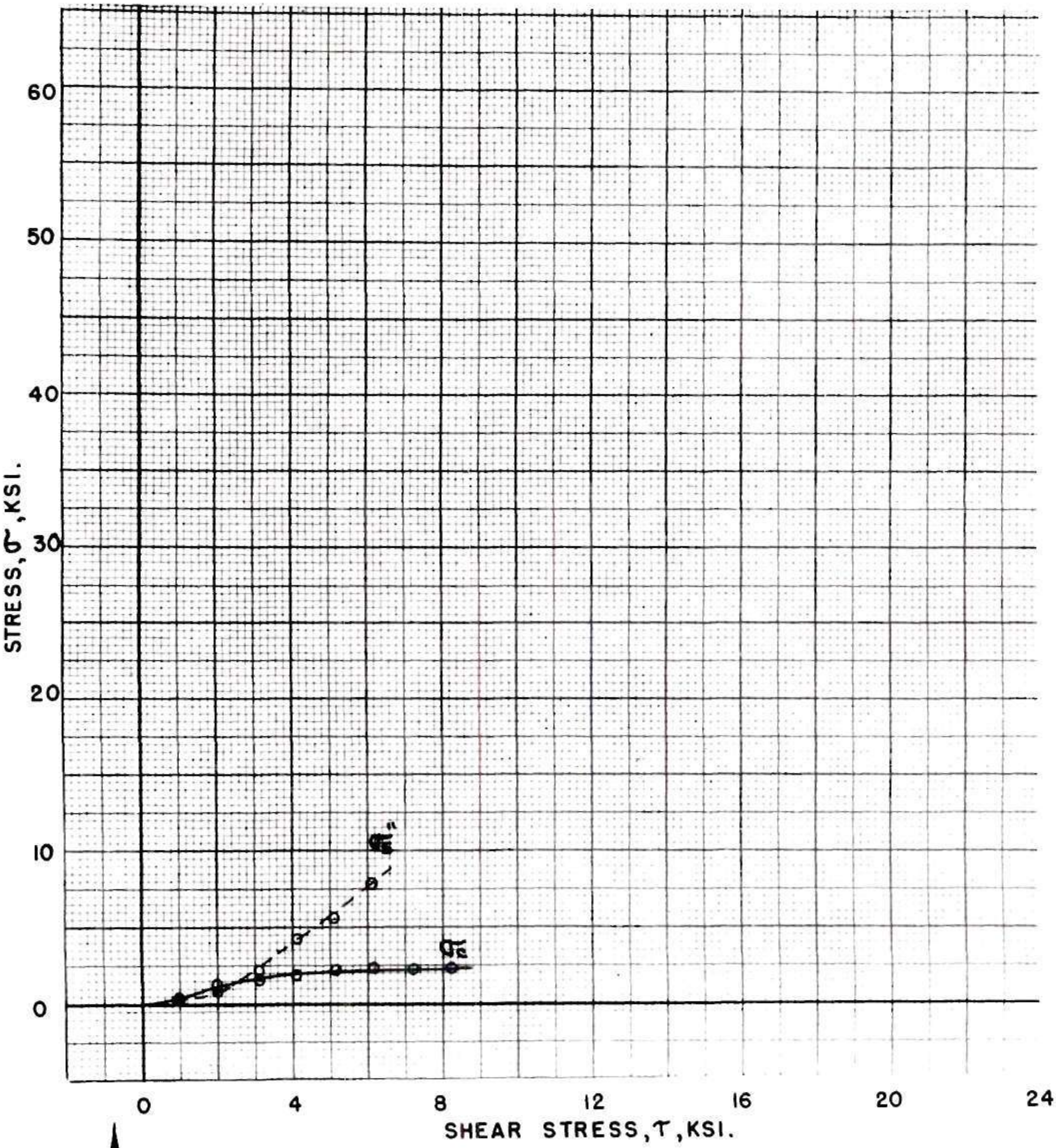


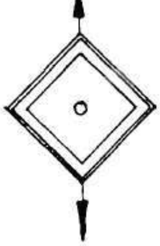
PRIMARY, σ_t, & SECONDARY, σ_s', STRESSES PARALLEL TO BUCKLE VS. SHEAR STRESS, τ.

.026 24ST

(PRESTRESSED)

Figure 12.





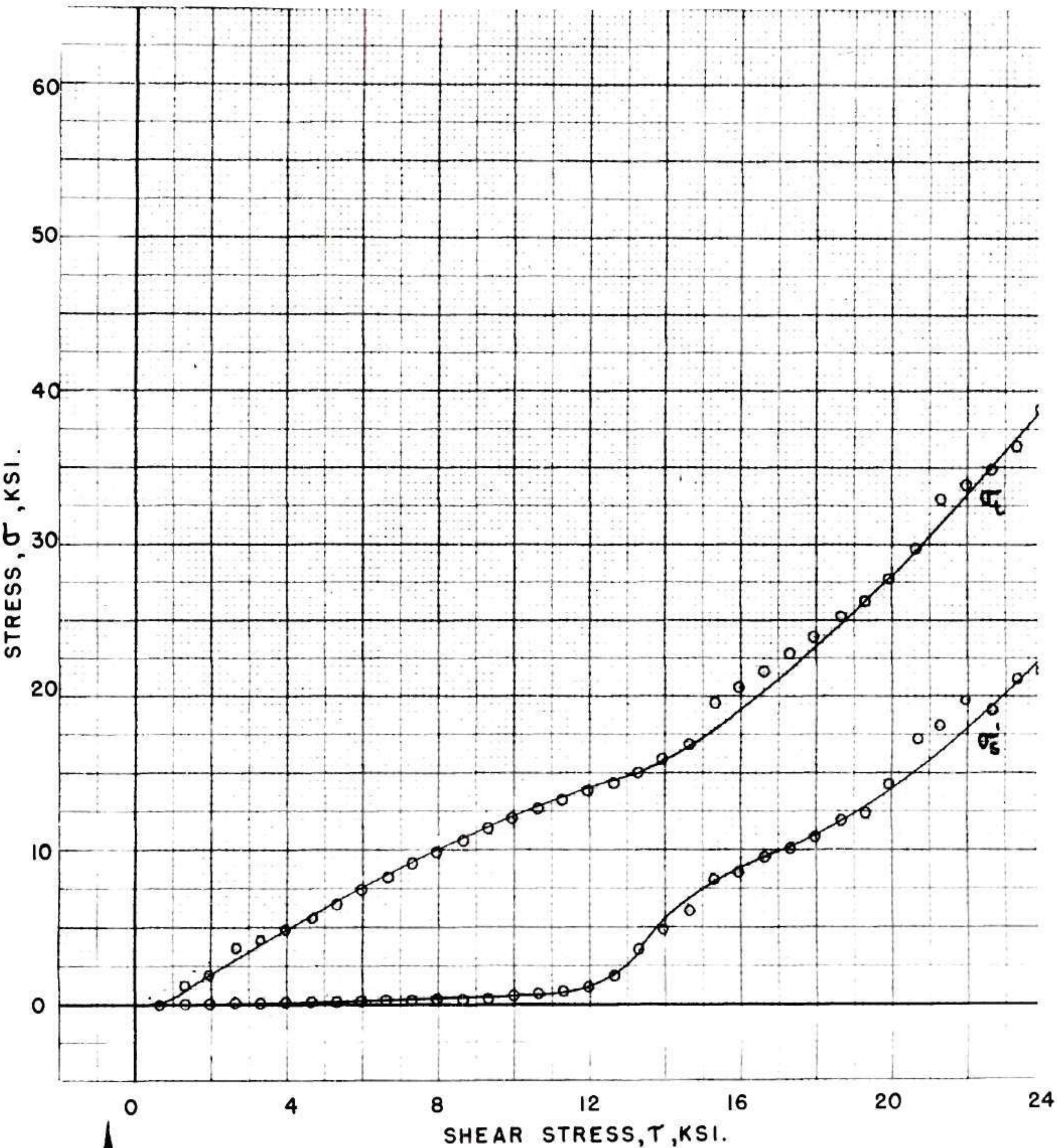
PANEL PROFILE & GAGE LOCATIONS

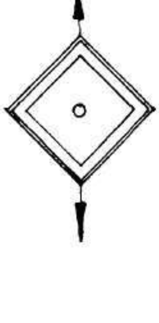
PRIMARY, σ_c, & SECONDARY, σ_s'', STRESSES PERPENDICULAR TO BUCKLE VS. SHEAR STRESS, τ.

.026 24ST

(PRESTRESSED)

Figure 13.

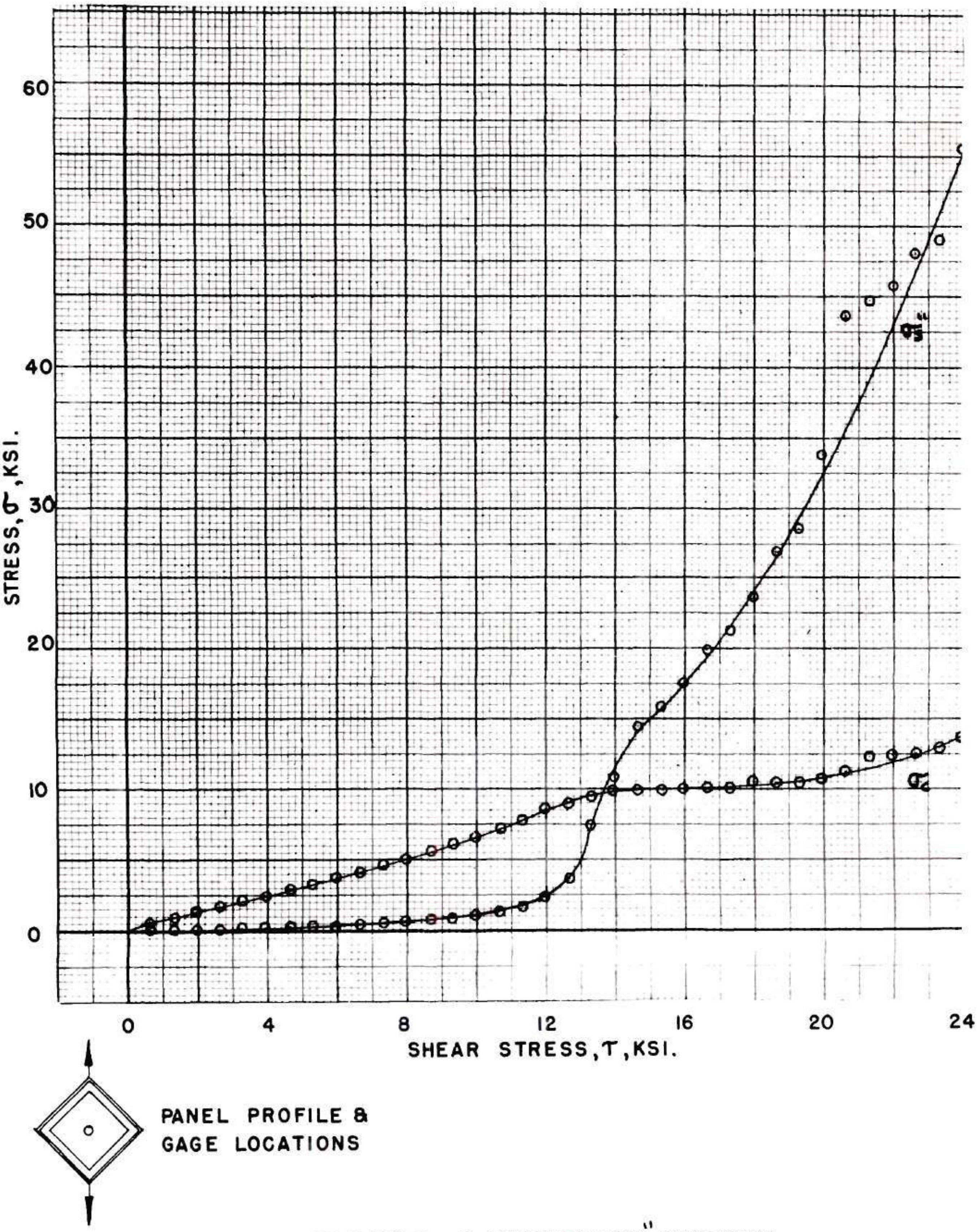




PRIMARY, σ_t, & SECONDARY, σ_s', STRESSES PARALLEL TO BUCKLE VS. SHEAR STRESS, τ.

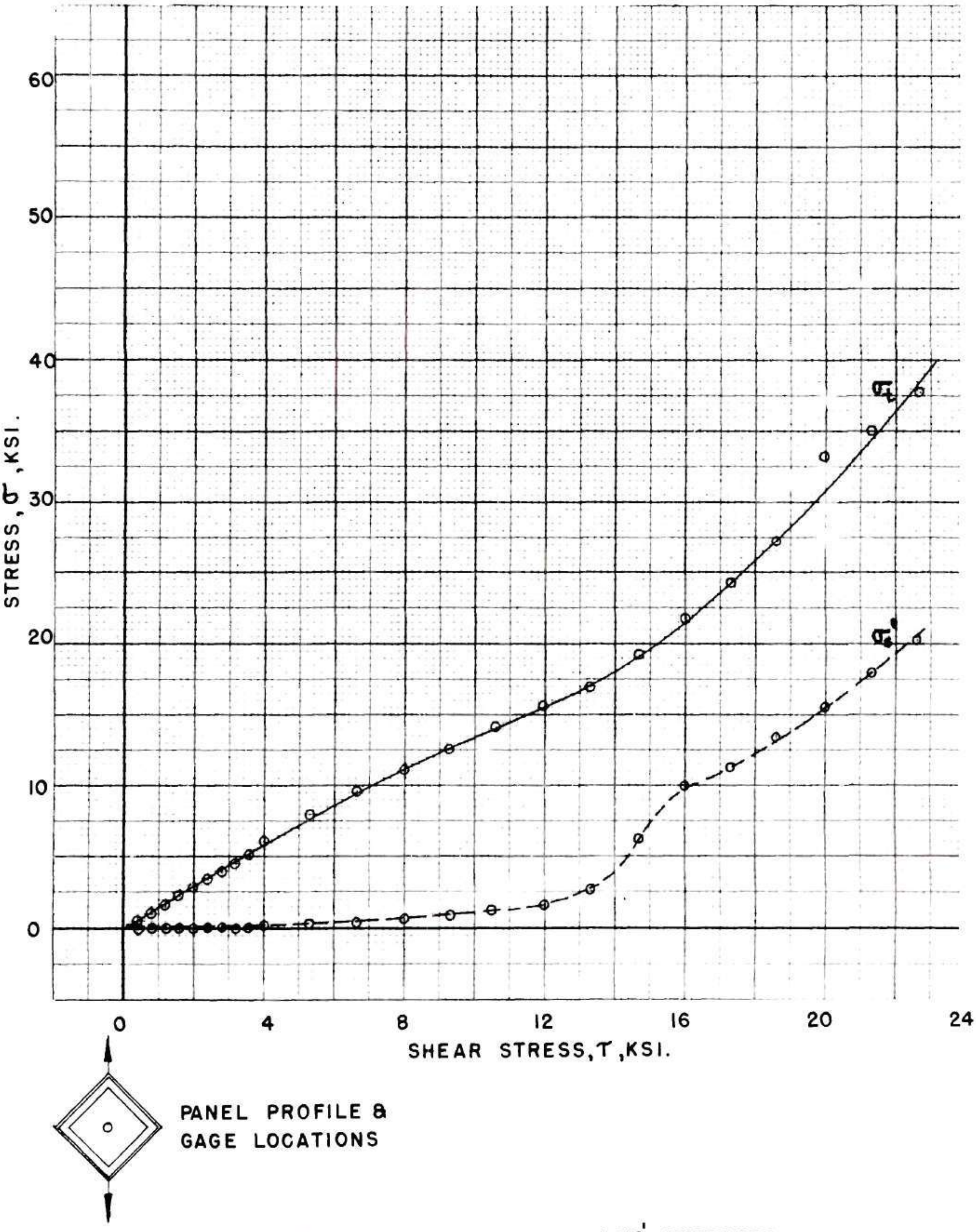
.067 24ST

Figure 14. Run No. 1



PRIMARY, σ_c, & SECONDARY, σ_s'', STRESSES
PERPENDICULAR TO BUCKLE VS. SHEAR STRESS, τ.
.067 24ST

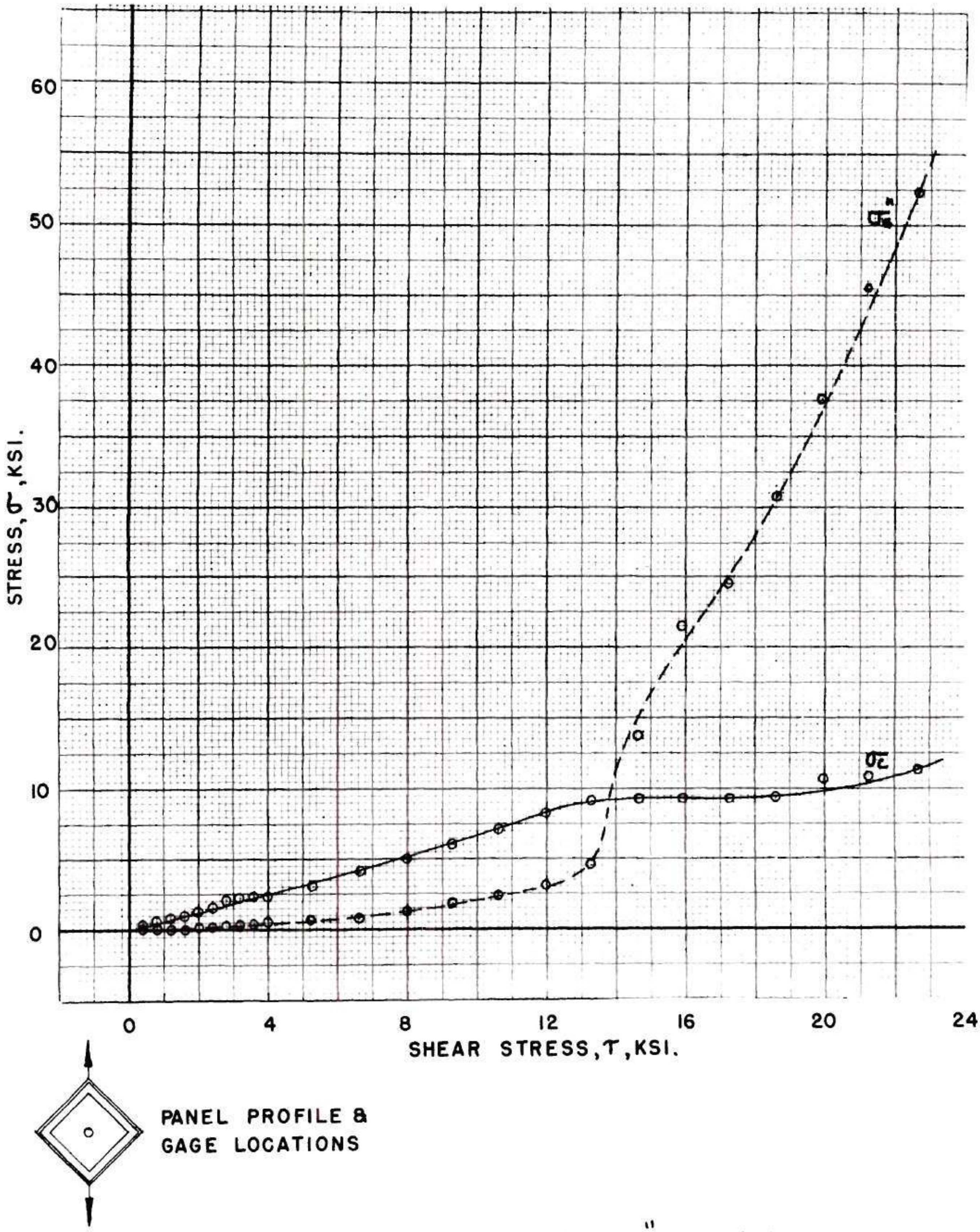
Figure 15. Run No. 1



PRIMARY, σ_t, & SECONDARY, σ_s', STRESSES PARALLEL TO BUCKLE VS. SHEAR STRESS, τ.

.067 24ST

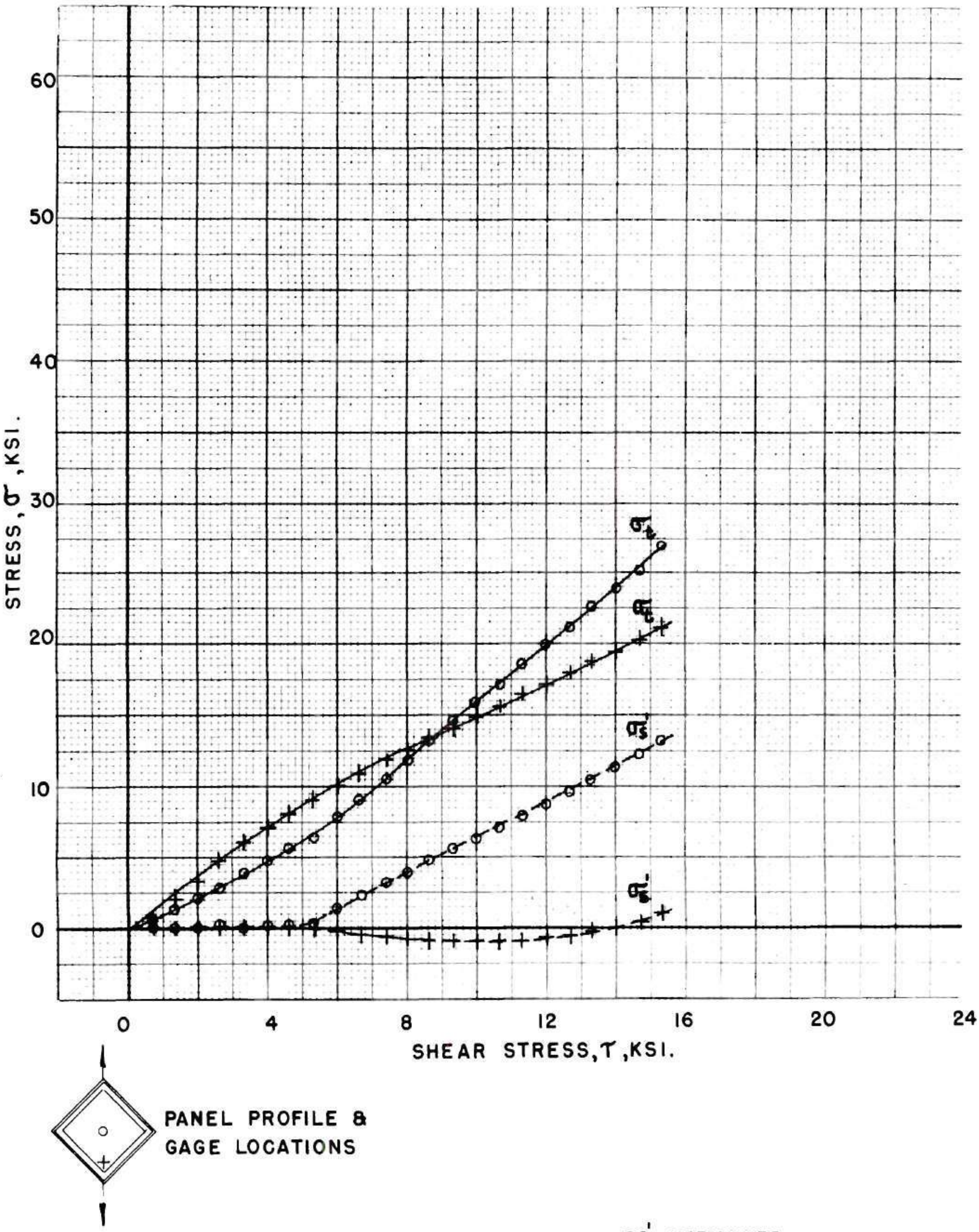
Figure 16. Run No. 2



PRIMARY, σ_c, & SECONDARY, σ_s'', STRESSES PERPENDICULAR TO BUCKLE VS. SHEAR STRESS, τ.

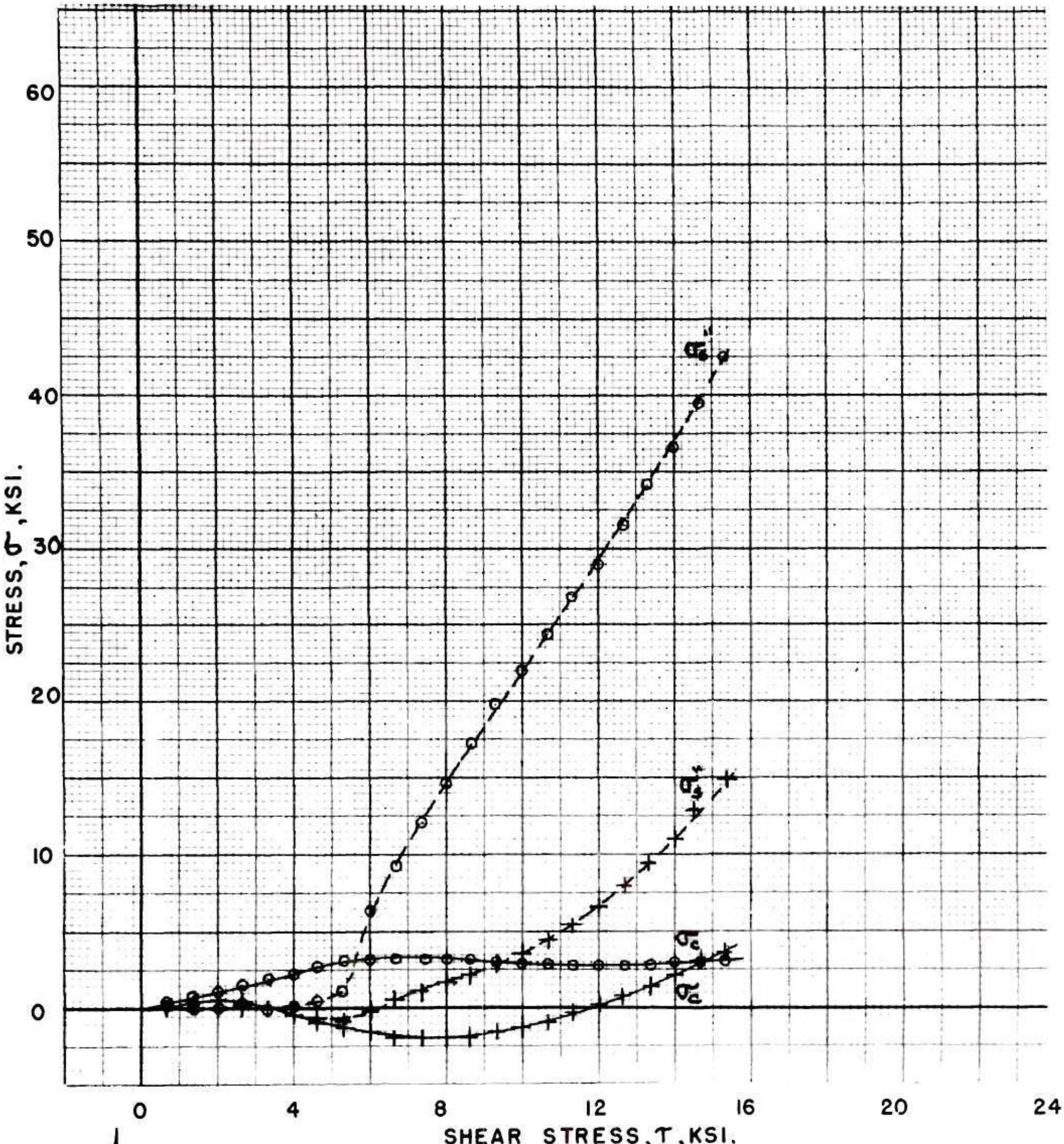
.067 24ST

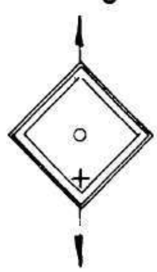
Figure 17. Run No. 2



PRIMARY, σ_t, & SECONDARY, σ_s', STRESSES
PARALLEL TO BUCKLE VS. SHEAR STRESS, τ.
.040 24ST

Figure 18.

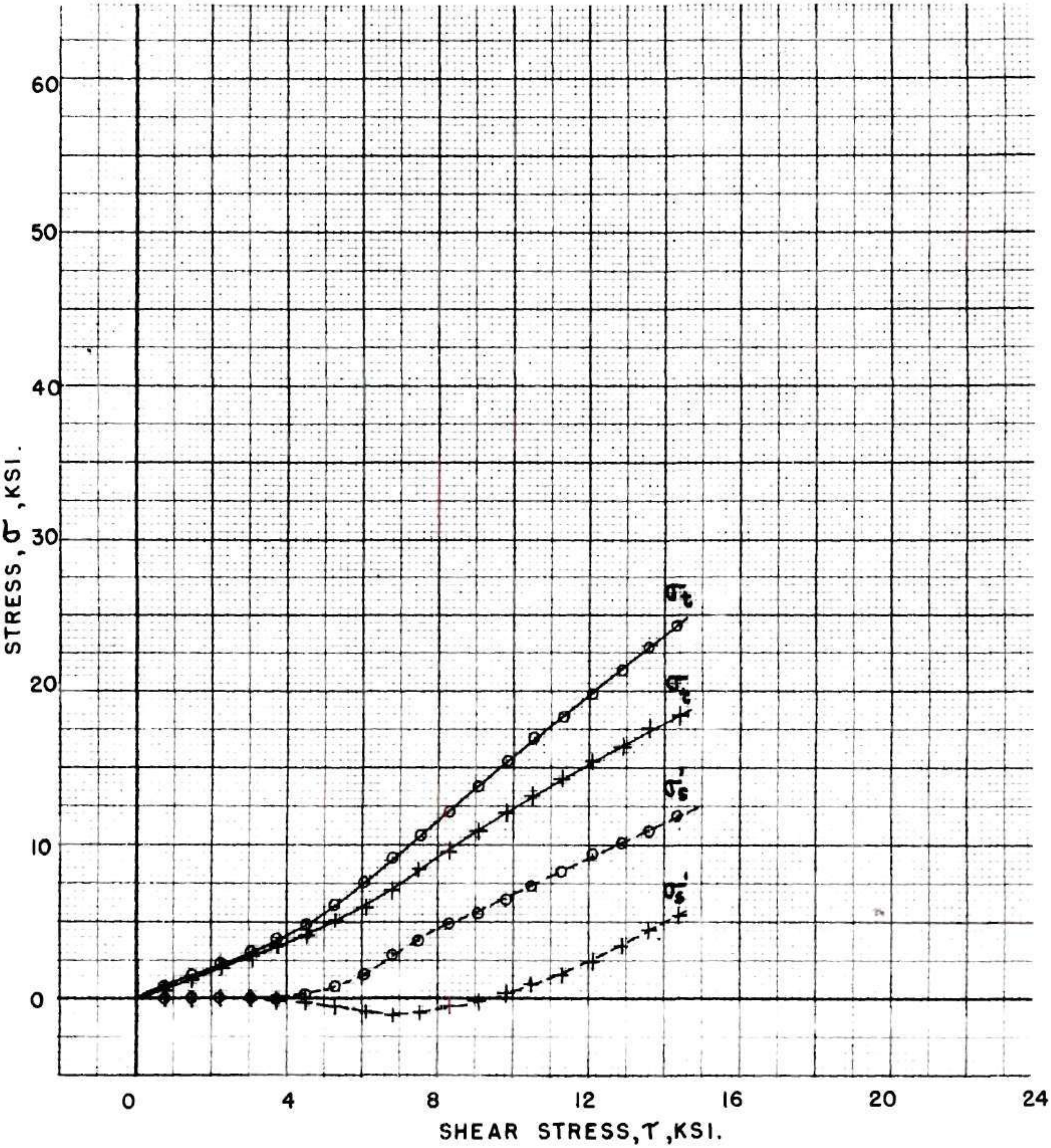


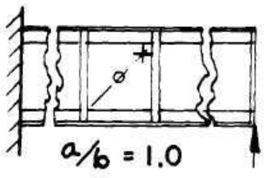


PANEL PROFILE &
GAGE LOCATIONS

PRIMARY, σ_c, & SECONDARY, σ_s'', STRESSES
PERPENDICULAR TO BUCKLE VS. SHEAR STRESS, τ.
.040 24ST

Figure 19.



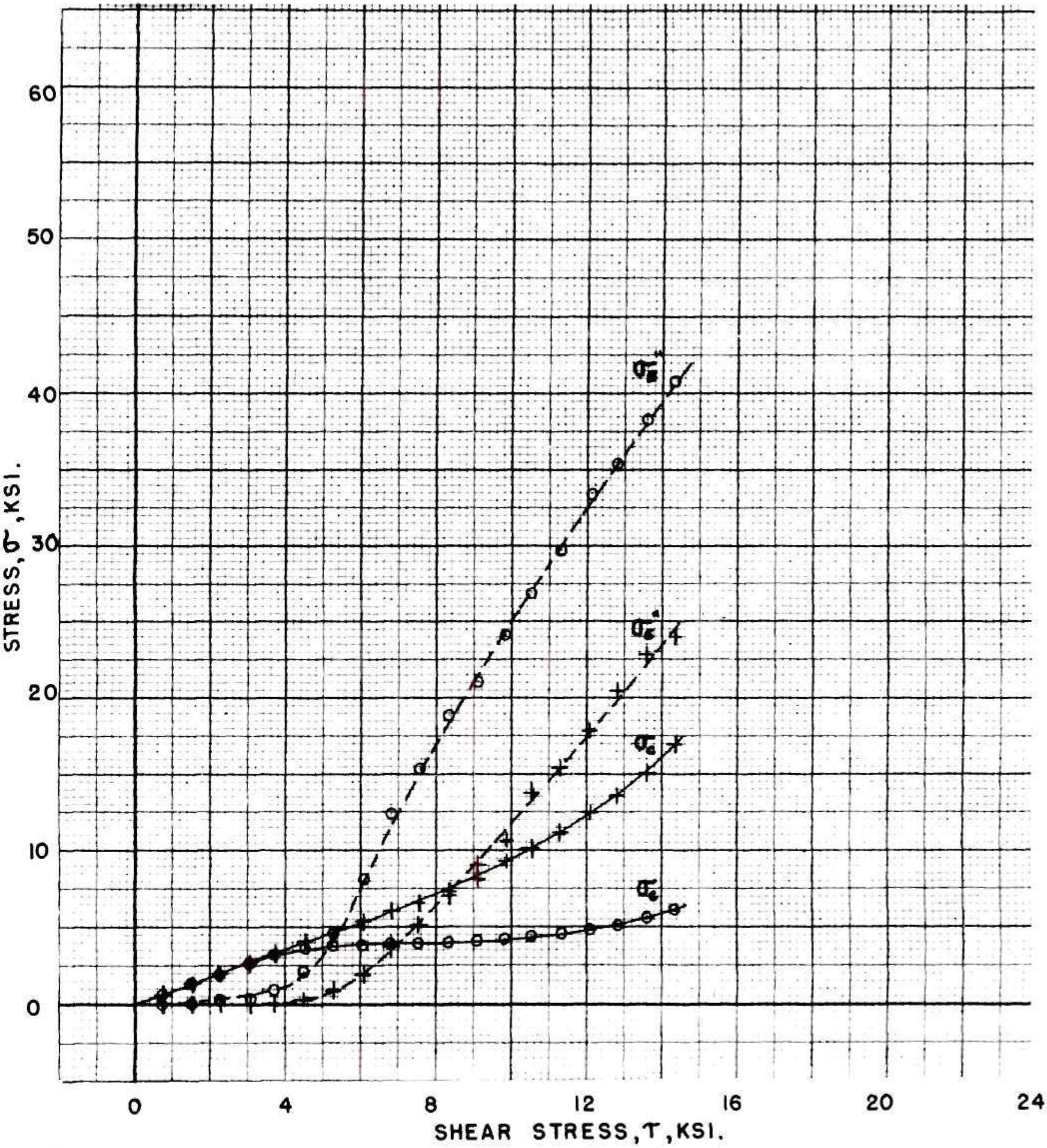


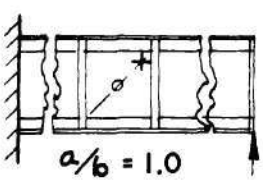
PANEL PROFILE & GAGE LOCATIONS

PRIMARY, σ_t, & SECONDARY, σ_s', STRESSES PARALLEL TO BUCKLE VS. SHEAR STRESS, τ.

.039 24ST

Figure 20. 1/8 Steel Cap Strips

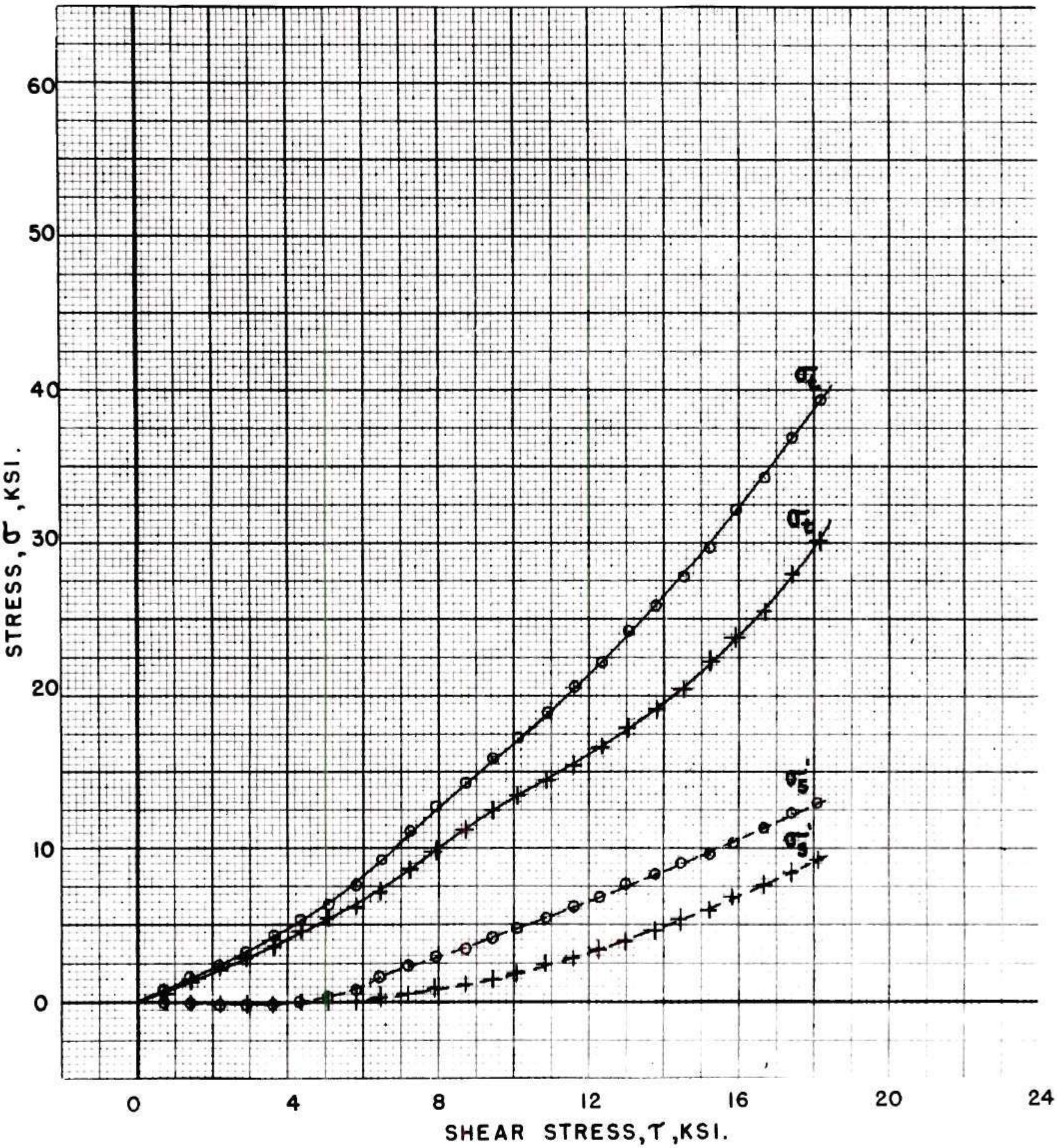


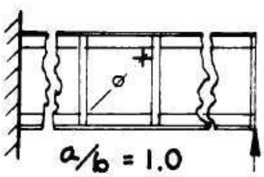


PANEL PROFILE & GAGE LOCATIONS

PRIMARY, σ_c, & SECONDARY, σ_s'', STRESSES
PERPENDICULAR TO BUCKLE VS. SHEAR STRESS, τ.
.039 24ST

Figure 21. 1/8 Steel Cap Strips



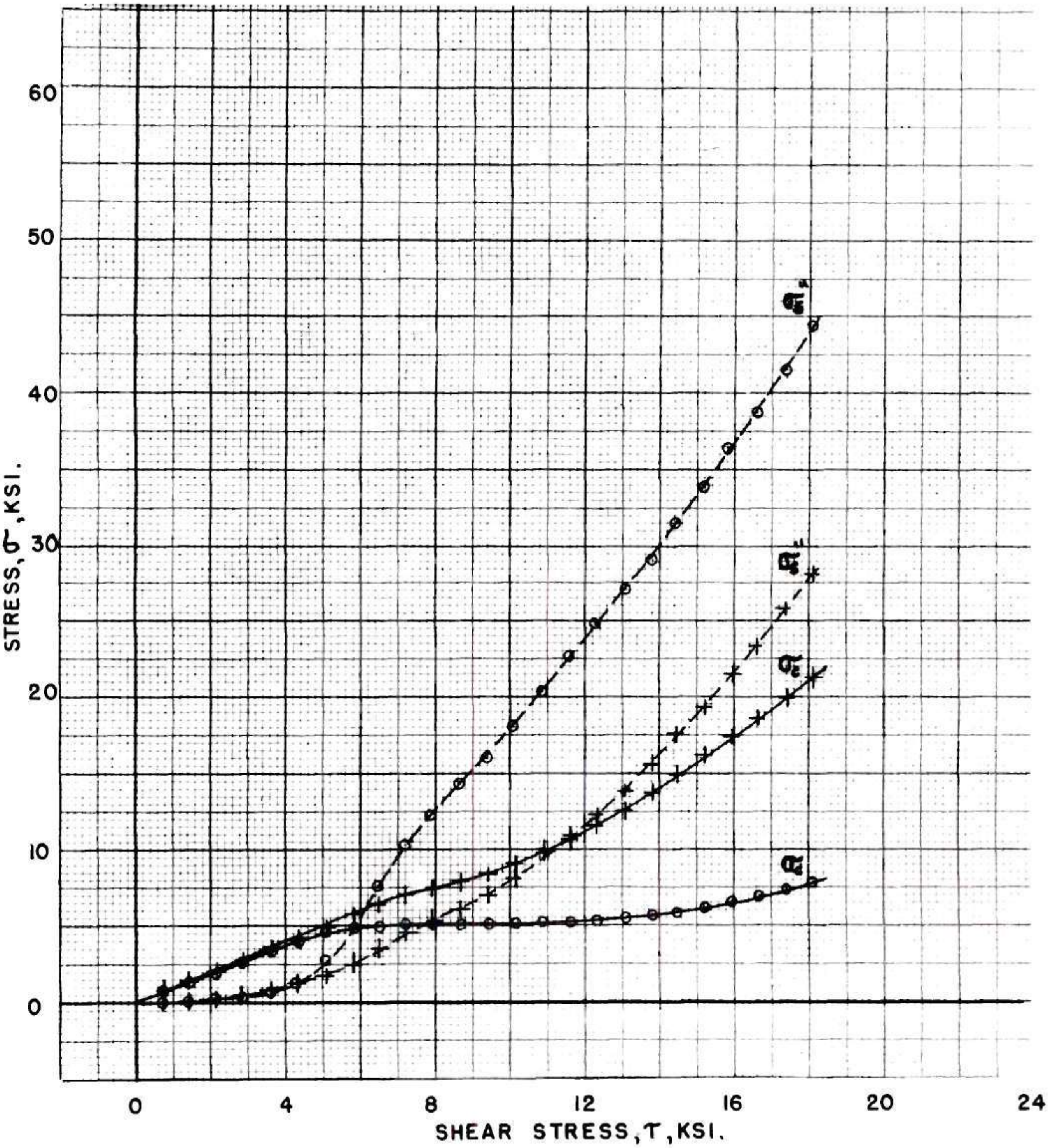


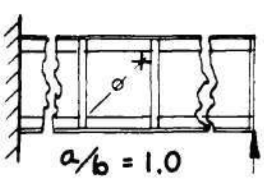
PANEL PROFILE & GAGE LOCATIONS

PRIMARY, σ_t, & SECONDARY, σ_s', STRESSES PARALLEL TO BUCKLE VS. SHEAR STRESS, τ.

.039 24ST

Figure 22.

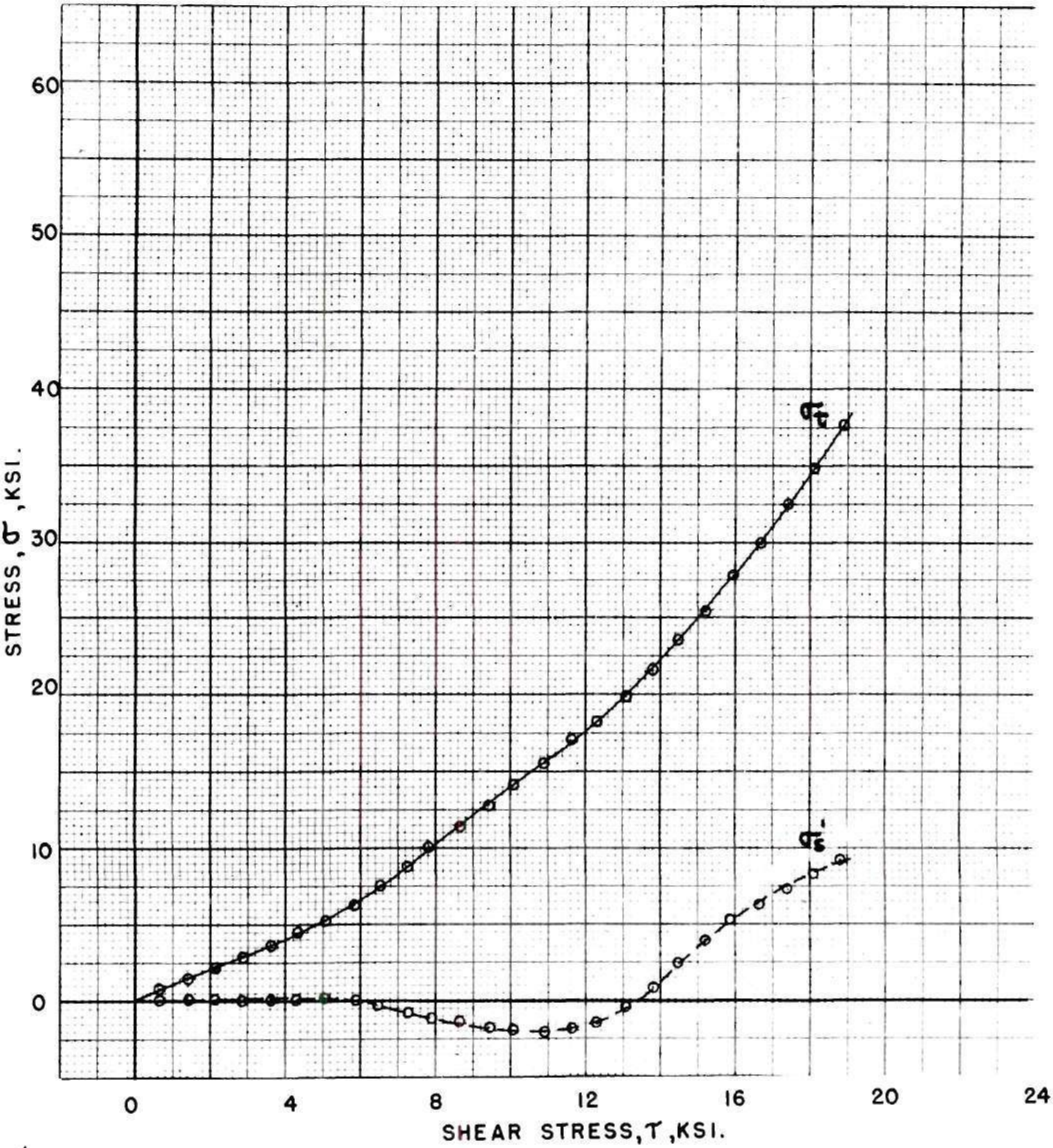


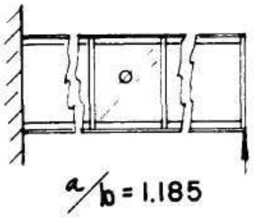


PANEL PROFILE & GAGE LOCATIONS

PRIMARY, σ_c, & SECONDARY, σ_s'', STRESSES PERPENDICULAR TO BUCKLE VS. SHEAR STRESS, τ.
.039 24ST

Figure 23.

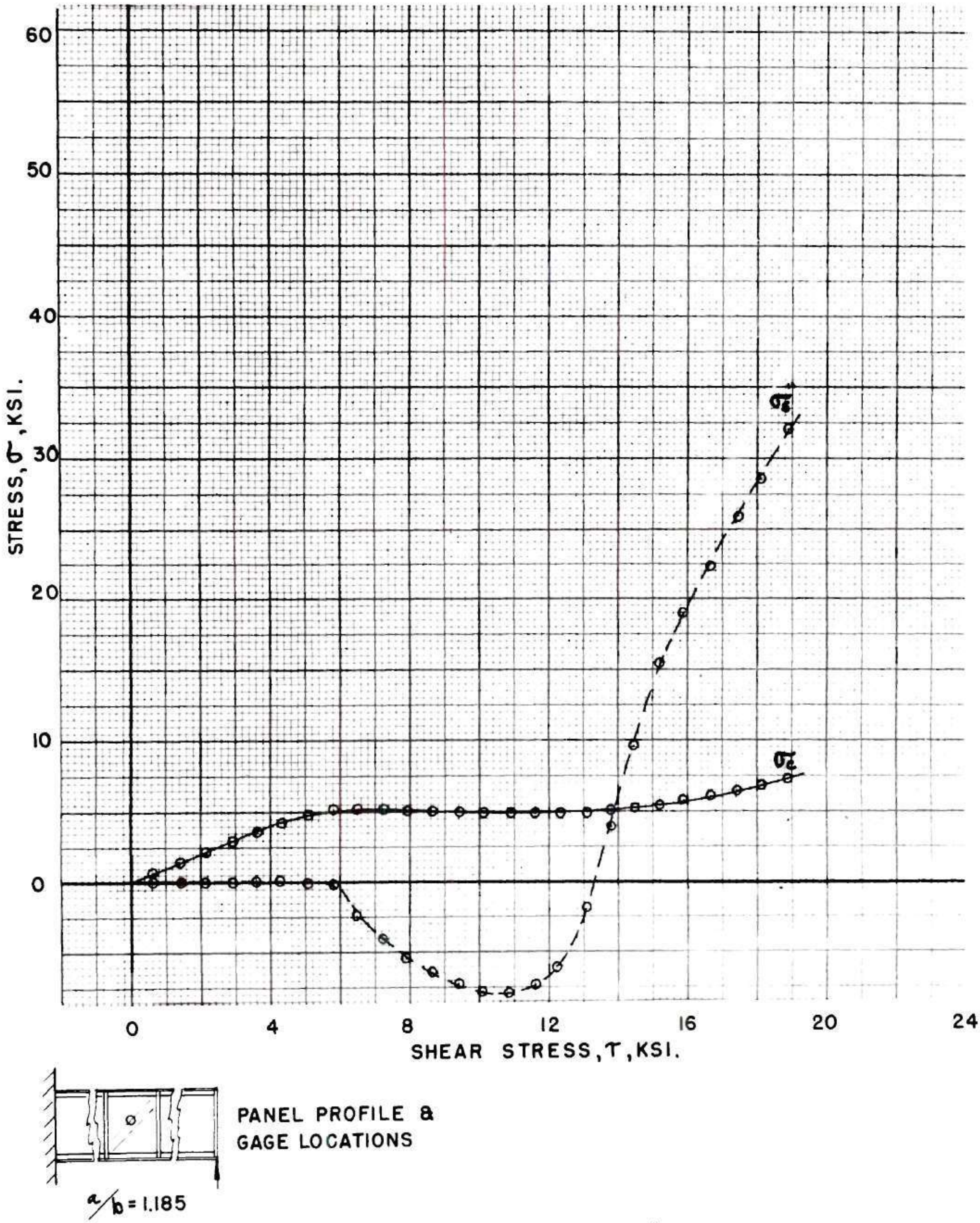




PRIMARY, σ_t, & SECONDARY, σ_s', STRESSES PARALLEL TO BUCKLE VS. SHEAR STRESS, τ.

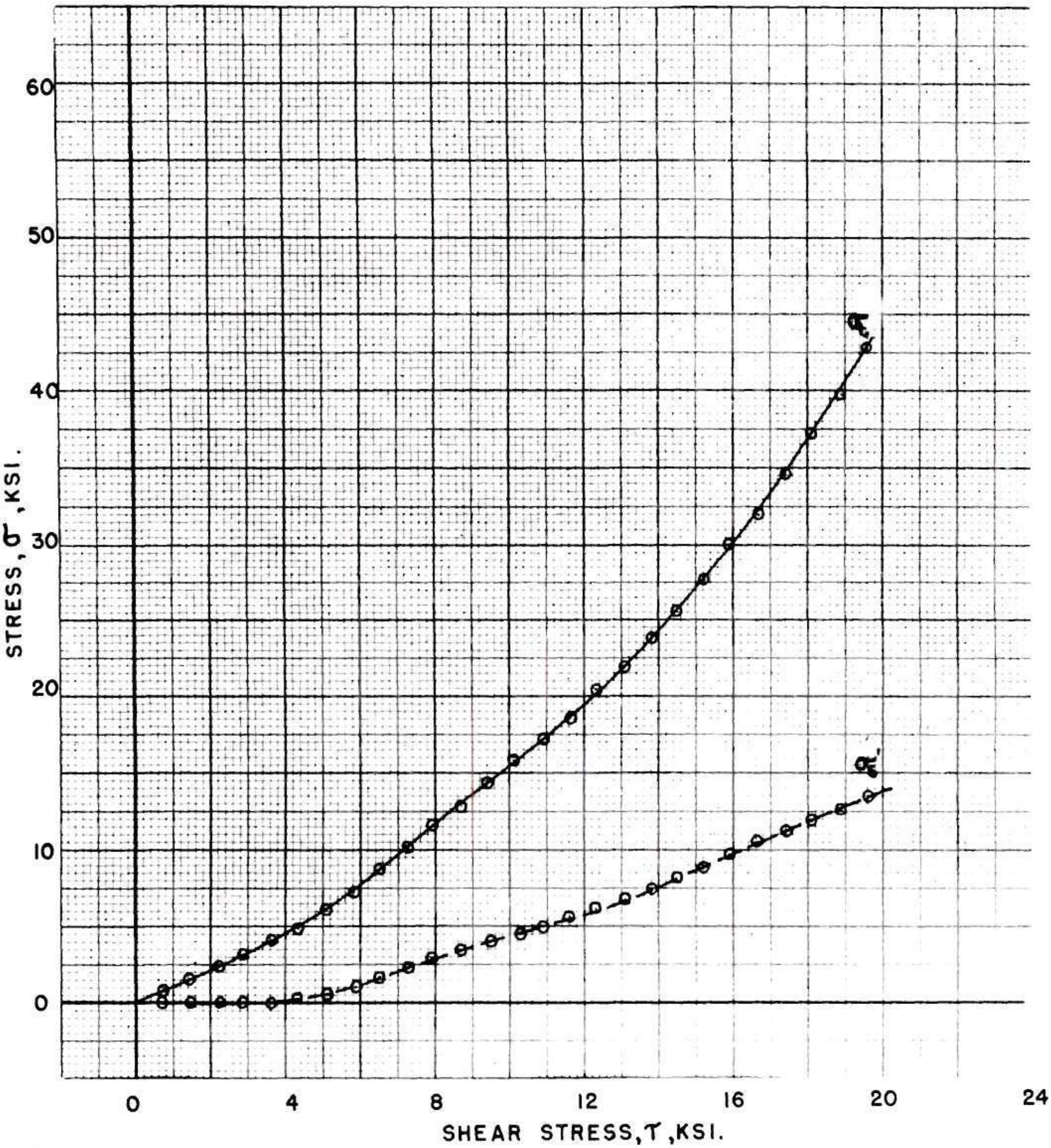
.039 24ST

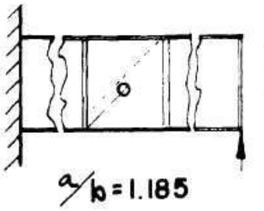
Figure 24. Run No. 1



PRIMARY, σ_c, & SECONDARY, σ_s'', STRESSES PERPENDICULAR TO BUCKLE VS. SHEAR STRESS, τ.
.039 24ST

Figure 25. Run No. 1

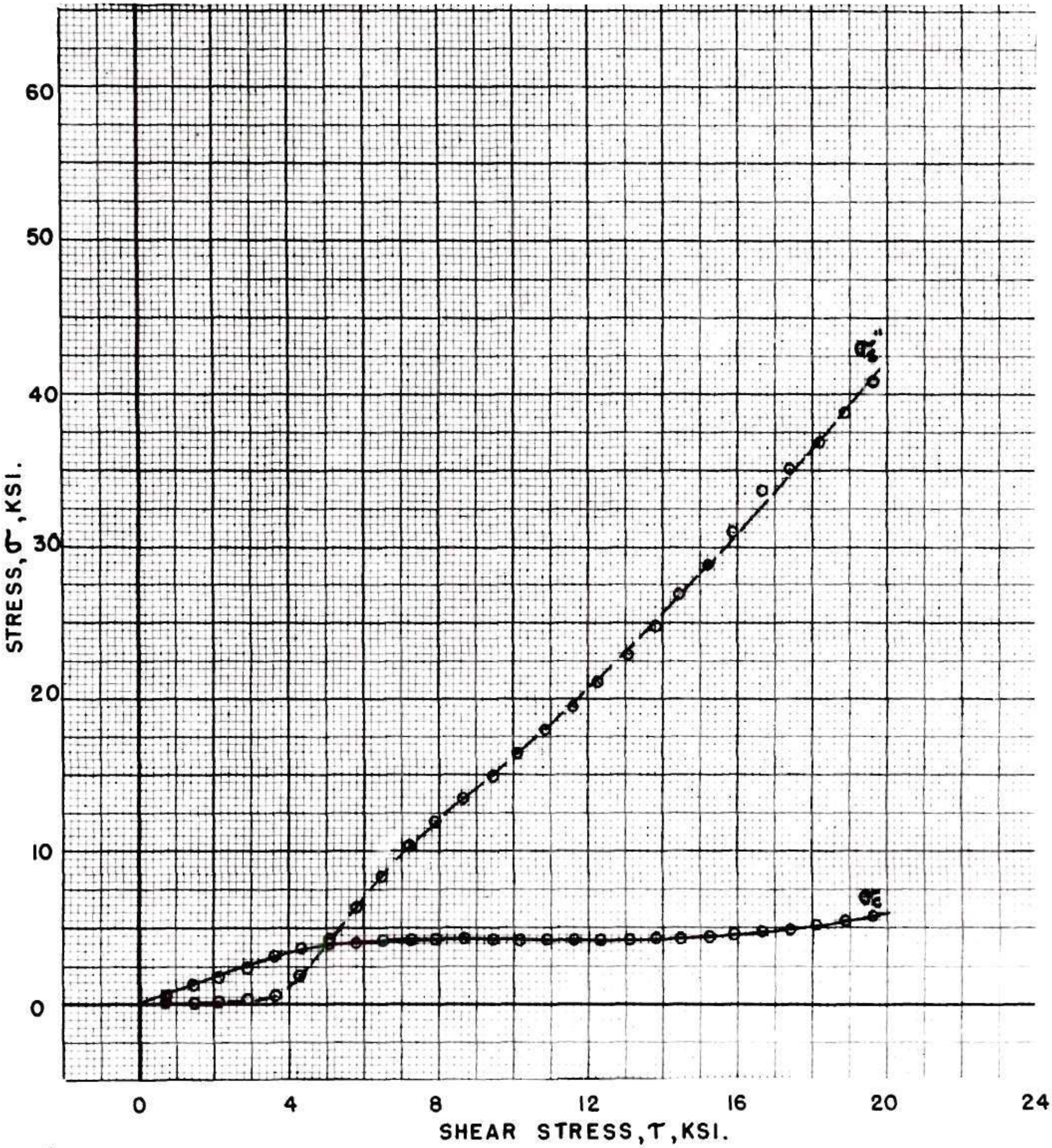


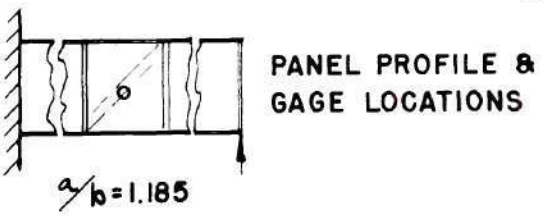


PRIMARY, σ_t, & SECONDARY, σ_s', STRESSES
PARALLEL TO BUCKLE VS. SHEAR STRESS, τ.

.039 24ST

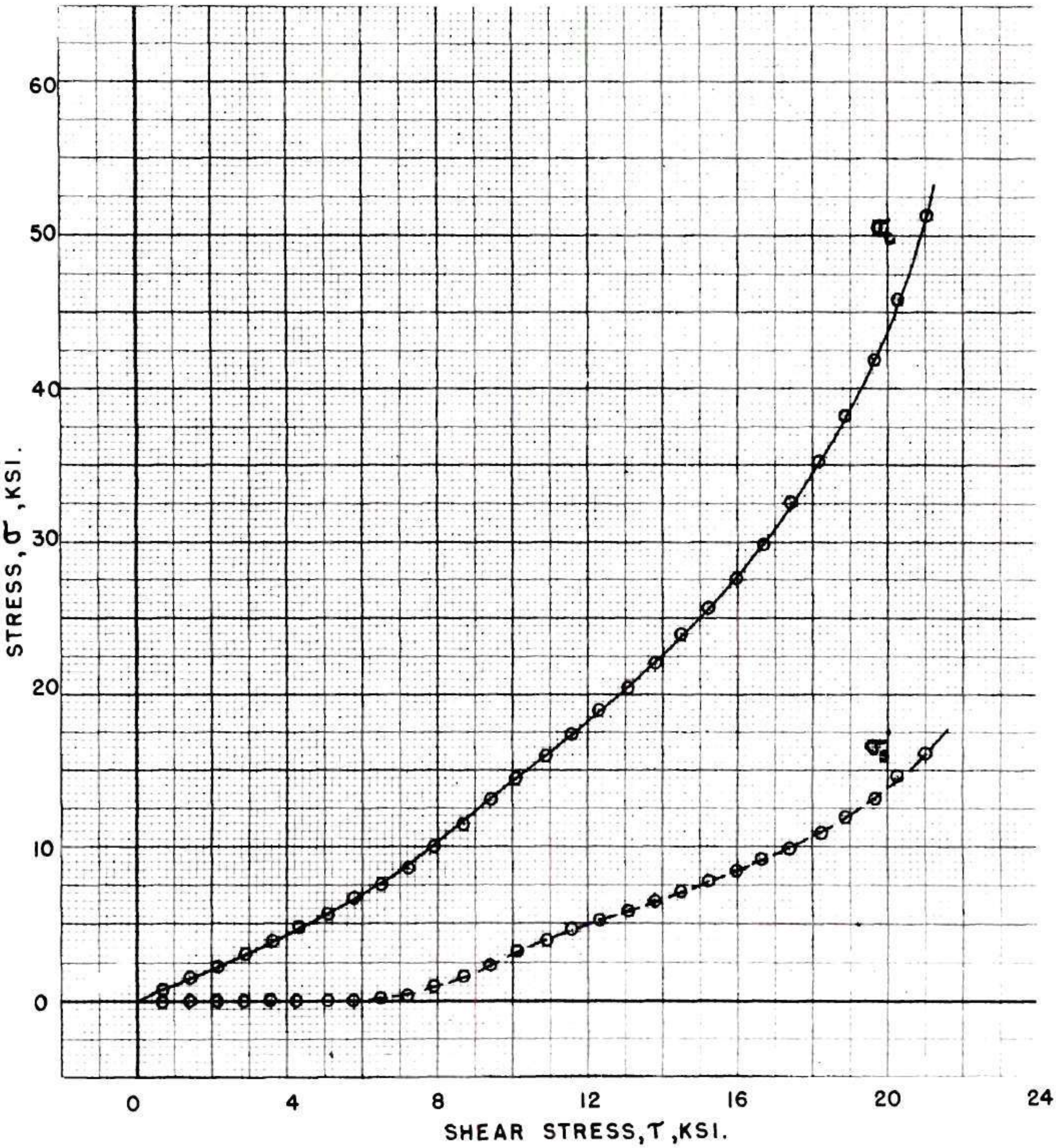
Figure 26. Run No. 2

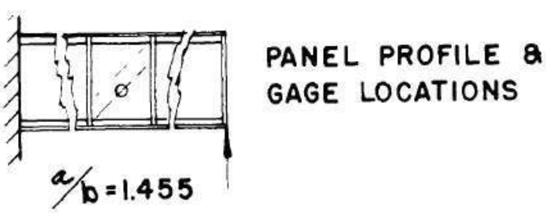




PRIMARY, σ_c, & SECONDARY, σ_s'', STRESSES
PERPENDICULAR TO BUCKLE VS. SHEAR STRESS, τ.
.039 24ST

Figure 27. Run No. 2

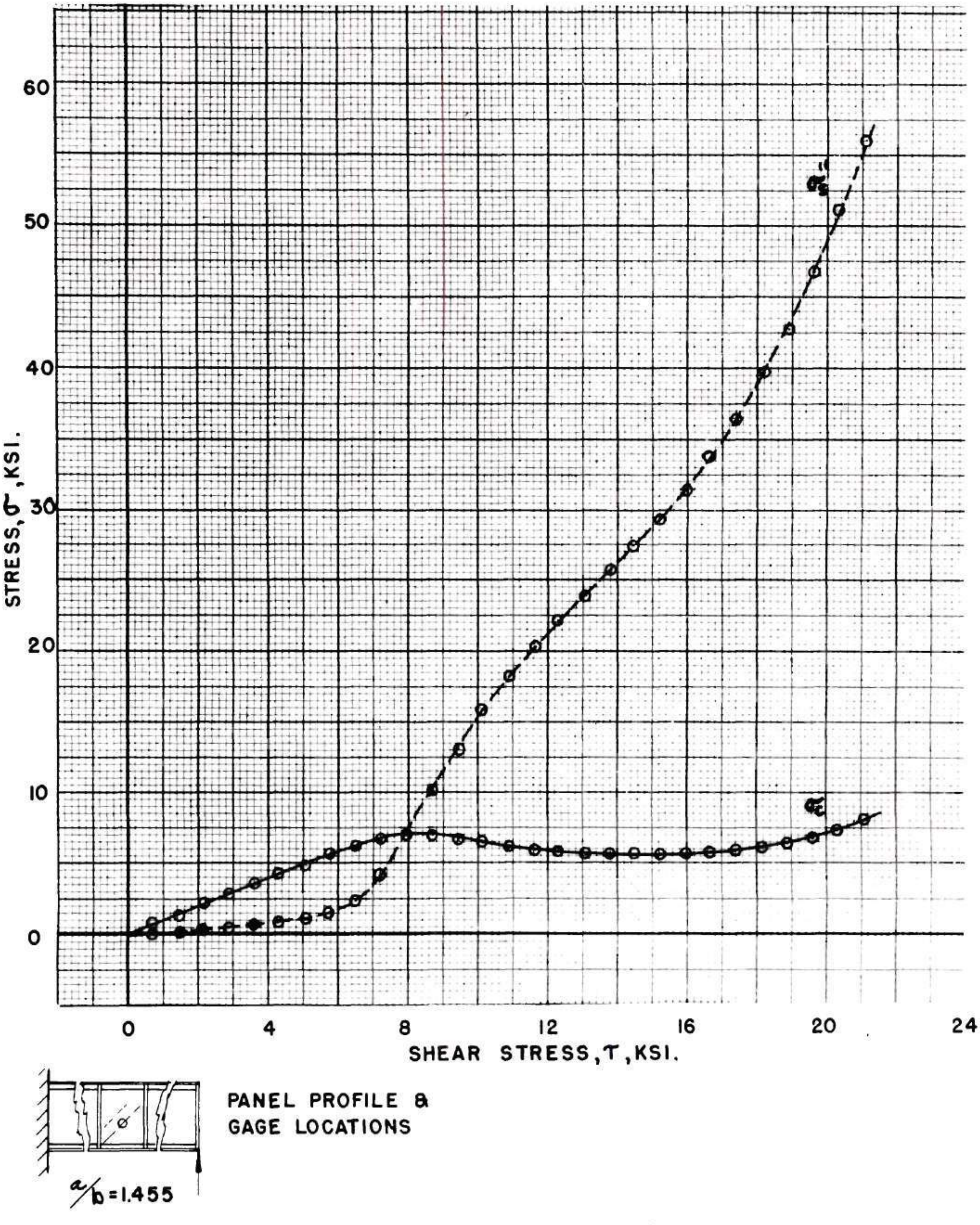




PRIMARY, σ_t, & SECONDARY, σ_s', STRESSES PARALLEL TO BUCKLE VS. SHEAR STRESS, τ.

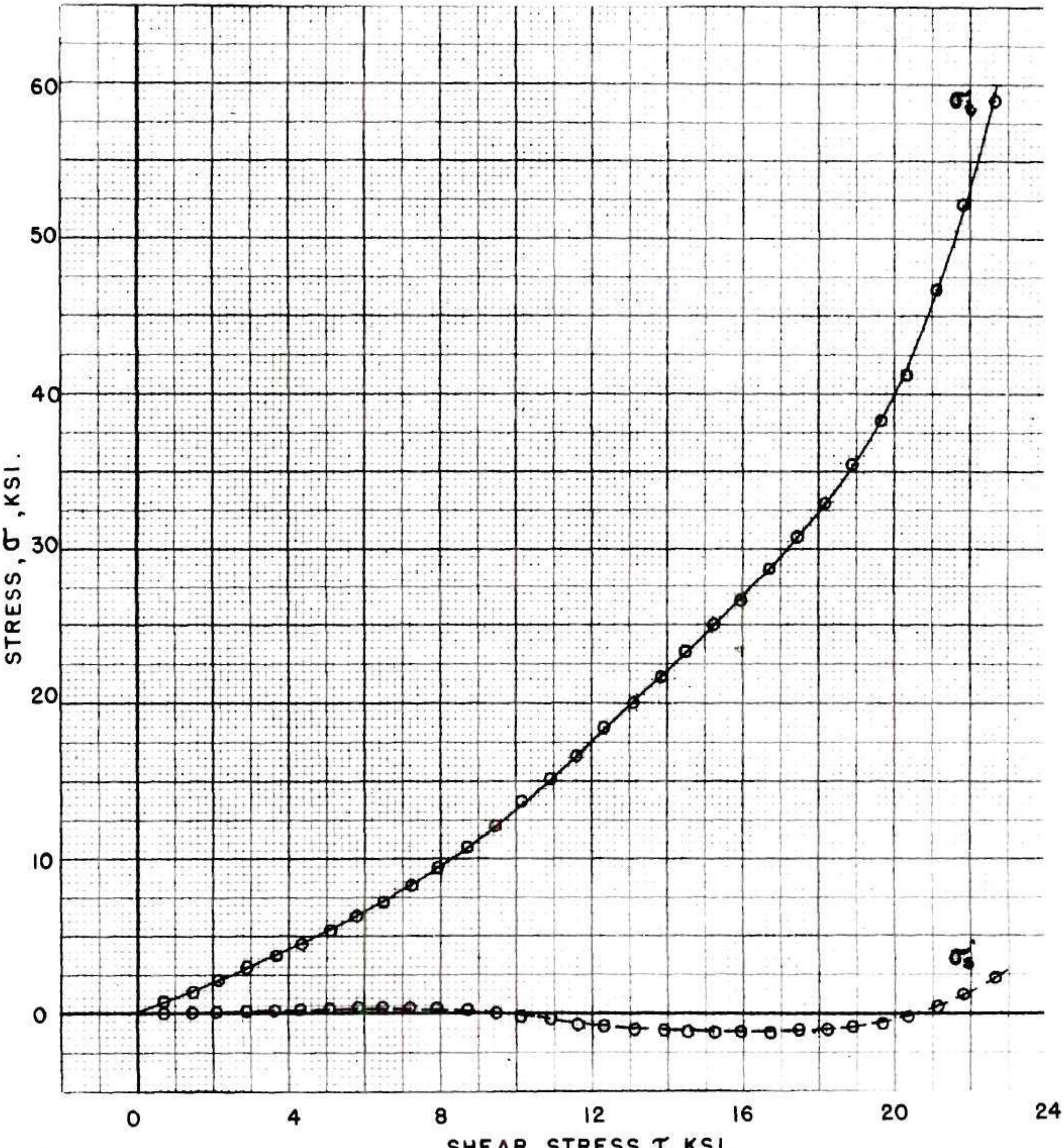
.039 24ST

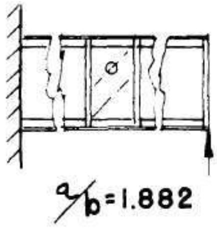
Figure 28.



PRIMARY, σ_c, & SECONDARY, σ_s'', STRESSES
PERPENDICULAR TO BUCKLE VS. SHEAR STRESS, τ.
.039 24ST

Figure 29.

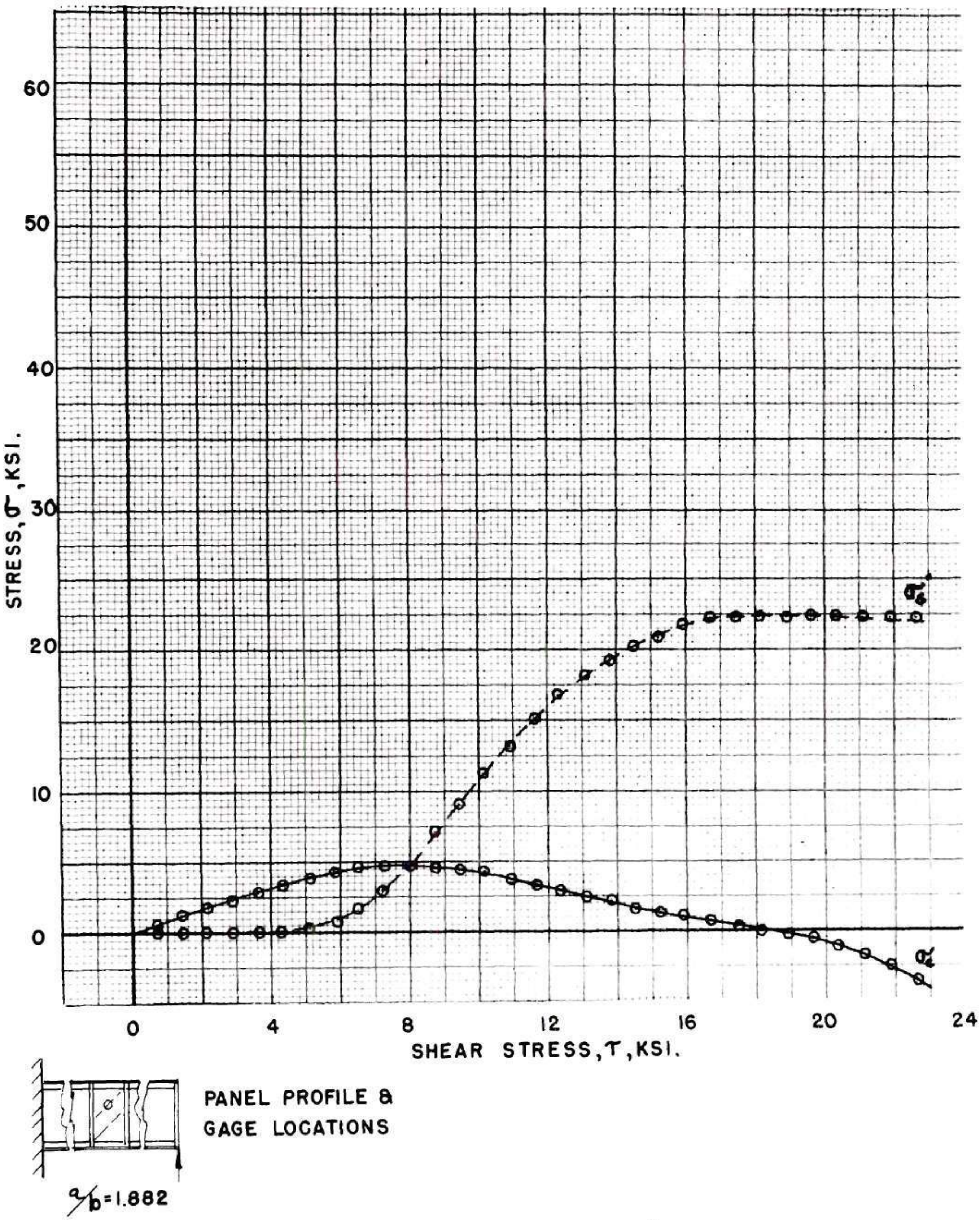




PANEL PROFILE & GAGE LOCATIONS

PRIMARY, σ_t', & SECONDARY, σ_s', STRESSES PARALLEL TO BUCKLE VS. SHEAR STRESS, τ.
.039 24ST

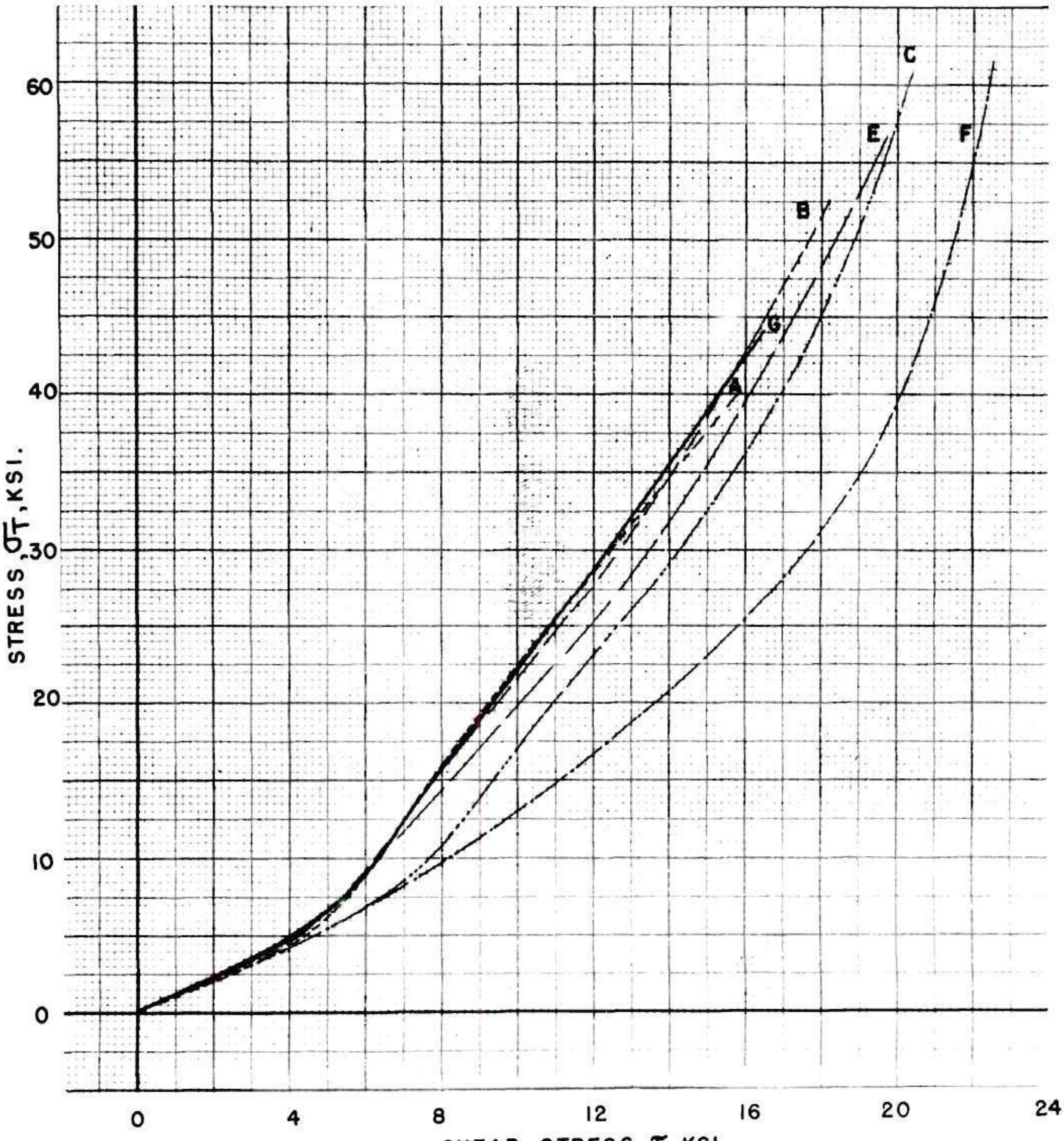
Figure 30.



PRIMARY, σ_c', & SECONDARY, σ_s'', STRESSES PERPENDICULAR TO BUCKLE VS. SHEAR STRESS, τ.

.039 24ST

Figure 31.



$a/b = 1.0$ PURE SHEAR

$a/b = 1.0$

$a/b = 1.0$

$a/b = 1.185$

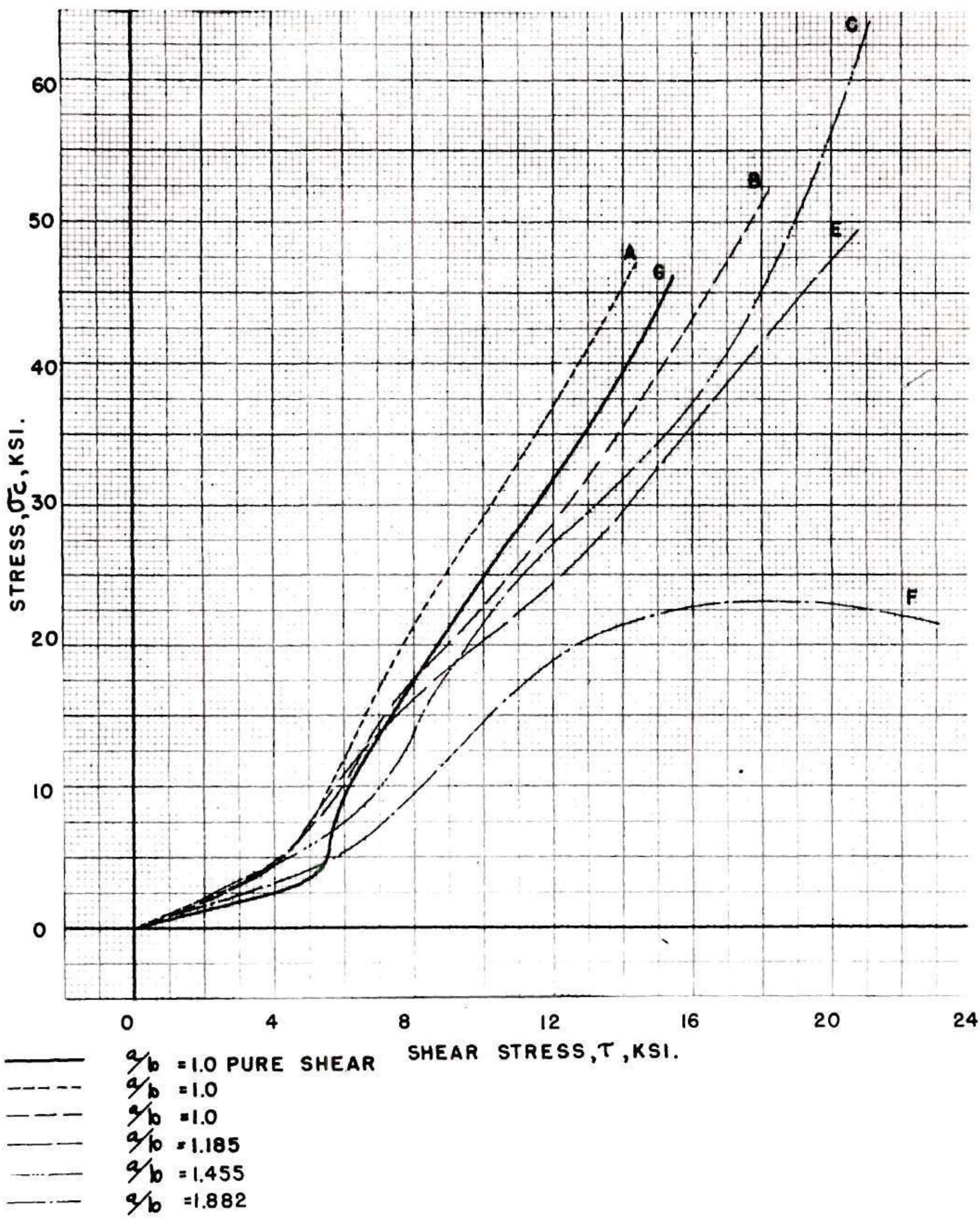
$a/b = 1.455$

$a/b = 1.882$

TOTAL OUTER FIBER STRESS, σ_T, PARALLEL TO BUCKLE VS. SHEAR STRESS, τ.

.040 NOMINAL WEB, 24ST

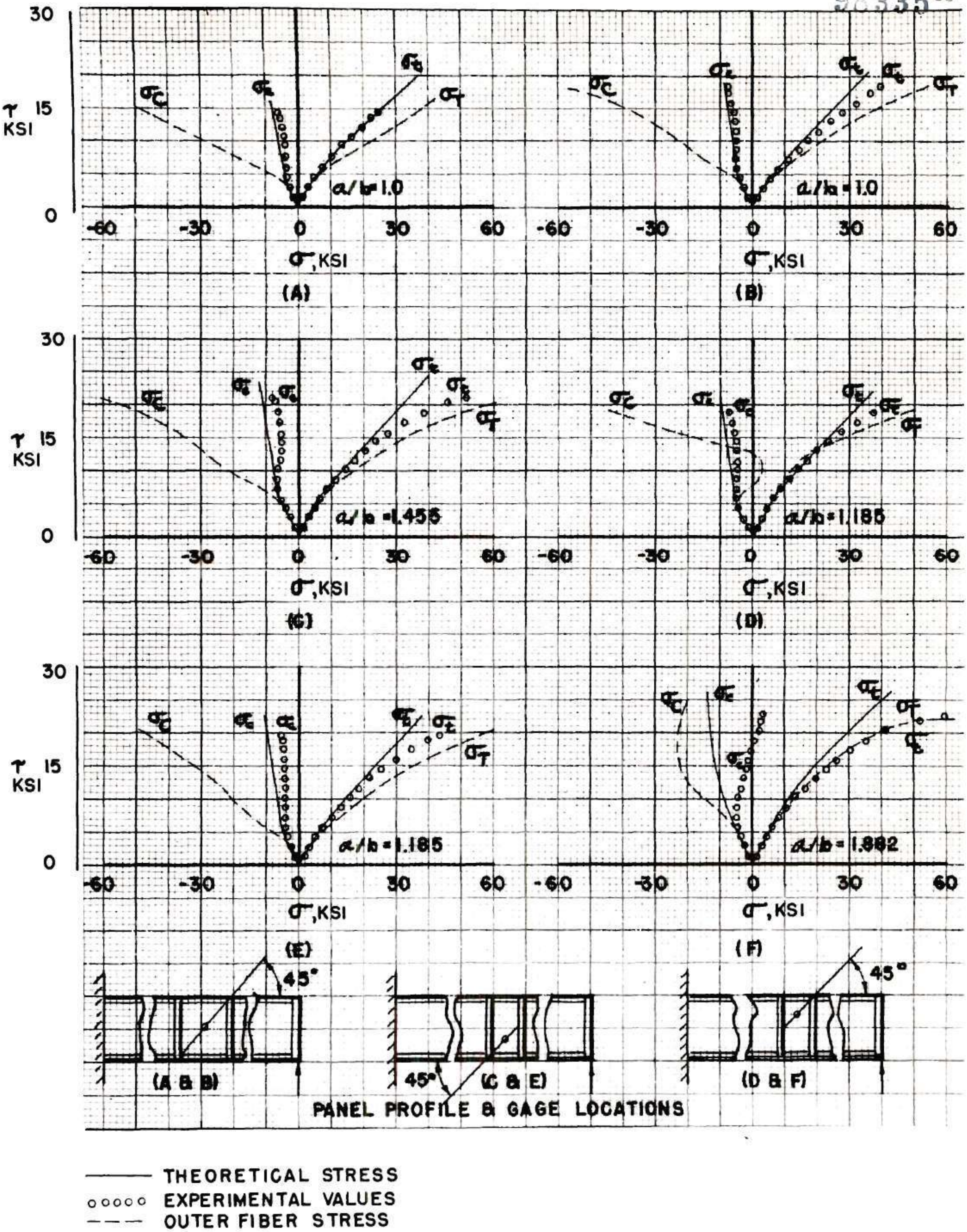
Figure 32.



TOTAL OUTER FIBER STRESS, σ_C,
PERPENDICULAR TO BUCKLE VS. SHEAR STRESS, τ.
.040 NOMINAL WEB, 24ST

Figure 33.

98335 60



WEB EXPERIMENTAL & THEORETICAL

STRESSES VS. SHEAR STRESS